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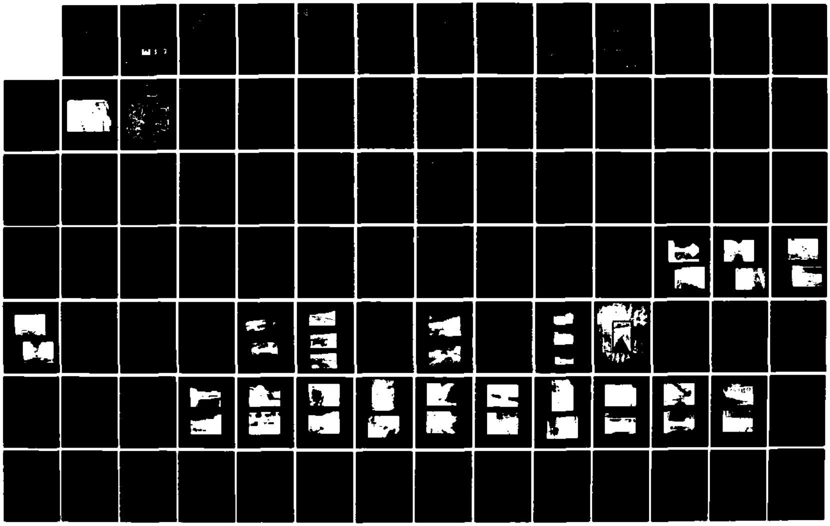

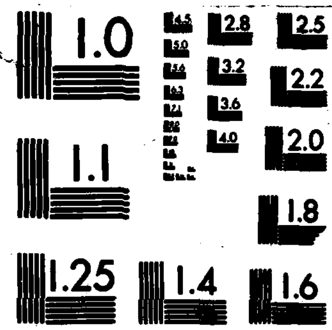

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AD-A156 488

MERRIMACK RIVER BASIN

PETERBOROUGH, NEW HAMPSHIRE

NOONE MILLS DAM

NH 00427

NHWRB NO. 191.02

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

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REPORT DOCUMENTATION PAGE

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18. SUPPLEMENTARY NOTES

Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Merrimack River Basin
Peterborough, New Hampshire
Contoocook River

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam is a stone-filled, concrete capped gravity overflow structure about 20 ft. high and 267 ft. long. The dam is considered to be in poor condition. It is small in size with a high hazard classification. There are a few major comcerns which must battended to. The potential for the loss of more than a few lives and extensive economic loss would exest.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:
NEDED-E

AUG 11 1980

Honorable Hugh J. Gallen
Governor of the State of New Hampshire
State House
Concord, New Hampshire 03301

Dear Governor Gallen:

Inclosed is a copy of the Noone Mills Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. The report is based upon a visual inspection, a review of past performance, and a preliminary hydrological analysis. A brief assessment is included at the beginning of the report.

The preliminary hydrologic analysis has indicated that the spillway capacity for the Noone Mills Dam would likely be exceeded by floods greater than 10 percent of the Probable Maximum Flood (PMF), the test flood for spillway adequacy. Our screening criteria specifies that a dam of this class which does not have sufficient spillway capacity to discharge fifty percent of the PMF, should be adjudged as having a seriously inadequate spillway and the dam assessed as unsafe, non-emergency, until more detailed studies prove otherwise or corrective measures are completed.

The term "unsafe" applied to a dam because of an inadequate spillway does not indicate the same degree of emergency as that term would if applied because of structural deficiency. It does indicate, however, that a severe storm may cause overtopping and possible failure of the dam, with significant damage and potential loss of life downstream.

It is recommended that within twelve months from the date of this report the owner of the dam engage the services of a professional or consulting engineer to determine by more sophisticated methods and procedures the magnitude of the spillway deficiency. Based on this determination, appropriate remedial mitigating measures should be designed and completed within 24 months of this date of notification. In the interim a detailed emergency operation plan and warning system should be promptly developed. During periods of unusually heavy precipitation, round-the-clock surveillance should be provided.

NEDED-E
Honorable Hugh J. Gallen

I have approved the report and support the findings and recommendations described in Section 7, with qualifications as noted above. I request that you keep me informed of the actions taken to implement these recommendations since this follow-up is an important part of the non-Federal Dam Inspection Program.

A copy of this report has been forwarded to the Water Resources Board, the cooperating agency for the State of New Hampshire, and the owner of the project, Darobsum Incorporated, 310 Marlboro Street, Keene, New Hampshire 03431.

Copies of this report will be made available to the public, upon request to this office, under the Freedom of Information Act, thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Water Resources Board for the cooperation extended in carrying out this program.

Sincerely,

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

NOONE MILLS DAM
NH 00427
NHWRB 191.02

MERRIMACK RIVER BASIN
PETERBOROUGH, NEW HAMPSHIRE

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

NATIONAL DAM INSPECTION PROGRAM
PHASE I - INSPECTION REPORT
BRIEF ASSESSMENT

Identification No: NH 00427

Name of Dam: Noone Mills Dam

Town: Peterborough

County and State: Hillsborough, New Hampshire

Stream: Contoocook River

Date of Inspection: December 11, 1979

Noone Mills Dam is a stone filled, concrete capped gravity overflow structure approximately 20 feet high from the top of the dam to the lowest ledge outcropping at the base and 267 feet long between abutments. The central portion of the dam consists of an overflow section about 102 feet long between training walls. The principal spillway is located at the right abutment and consists of a 34 feet long concrete spillway with flashboards. There are two potential low level outlets through the dam including a 4.0 feet wide by 4.5 feet high sluice gate opening located to the left of the principal spillway and a 5.5 feet diameter penstock intake located at the left abutment. However, both gates are currently inoperable. There is no emergency spillway.

The dam impounds water from the Contoocook River, which after passing over the spillway and overflow section continues to flow in a northeasterly direction through the center of Peterborough. The purpose of the dam is to provide water for industrial purposes at the adjoining mill. The impoundage is about 0.31 miles in length with a surface area of about 19 acres. The maximum storage capacity is about 315 acre feet.

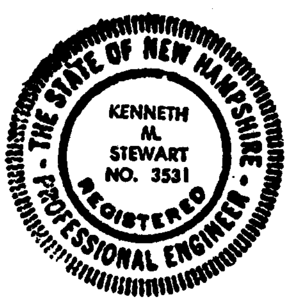
As a result of the visual inspection and the review of available data regarding this facility, the dam is considered to be in POOR condition. Major concerns are: a severe cavity in the concrete at the base of the buttress forming the right training wall of the overflow section; severe erosion and cracking of concrete structures in numerous locations; major seepage at the toe of the left embankment wall near the left training wall; the leakage and inoperable condition of both low level outlets; and the inadequacy of the spillway and overflow section to pass the test flood.

This dam is classified as SMALL in size and a HIGH hazard structure in accordance with the recommended guidelines established by the Corps of Engineers. The test flood for this dam, therefore, ranges from one-half the Probable Maximum Flood (1/2 PMF) to the Probable Maximum Flood (PMF). Since the dam falls on the lower end of the small size range, the 1/2 PMF was utilized for this hydrologic analysis. The test flood inflow was estimated to be 38,200 cfs and resulted in a routed test flood outflow equal to 37,600 cfs which would overtop the dam crest by about 6.7 feet. The maximum combined spillway and overflow section discharge capacity with the water level at the dam crest was estimated to be 7,400 cfs or about 20 percent of the routed test flood outflow.

An assumed failure of the Noone Mills Dam with the water surface at the dam crest would increase the stage along the immediate downstream channel to about 12 feet, more than 3 feet above the stage of the prefailure tailwater. This would result in a water depth of approximately 8 to 9 feet above the sill of the lower floor of the mill building located adjacent to the western side of the downstream channel, and reach the un-bricked portions of the mill building windows. In addition to this, the foundation of the building located immediately downstream from the left abutment of the dam could be undermined, causing extensive damage to this structure. Approximately one-half mile downstream, the channel cross-section broadens and the stage would be reduced to just over 6 feet which is only about a one foot increase in stage above prefailure tailwater. However, as the channel approaches the Route 101 bridge, approximately 1 mile downstream, it has a lesser slope and more steeply sloped banks, which present a small cross-section available to accommodate the discharge. Consequently, the stage in this stretch of the river would increase to more than 12 feet in order to pass the discharge, which is more than 2 feet above the stage of the prefailure tailwater. The prefailure flow would reach the parking lot and approach the sill of the shopping center buildings. The increase in stage resulting from the failure discharge would cause water to rise to above the sill of the shopping center buildings and impact light industrial buildings located near the shopping center. The potential for the loss of more than a few lives and extensive economic loss would exist.

It is recommended that the owner engage a qualified registered engineer to investigate and design remedial measures for the severe cavity in the concrete at the base of the buttress forming the right training wall of the overflow section; to investigate and design remedial measures for the severe erosion and cracking of concrete structures in numerous locations; to investigate the major seepage at the toe of the left embankment wall near the left training wall; to investigate and design remedial measures for the leakage and inoperable condition of both low level outlets; and to do a detailed hydrologic-hydraulic investigation to assess further the potential of overtopping the dam, the adequacy of the spillway and overflow section to pass the test flood, and the need for and means to increase project discharge capacity. It is also recommended that the owner remove the trees, brush and debris from the immediate area of the dam.

The recommendations and remedial measures are described in Section 7 and should be addressed by the owner within one year after receipt of this Phase I Inspection Report.



Kenneth M. Stewart
Kenneth M. Stewart
Project Manager
N.H.P.E. 3531

S E A Consultants Inc.
Rochester, New Hampshire

This Phase I Inspection Report on Noone Mills Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

ARAMAST MAHTESIAN, MEMBER
Geotechnical Engineering Branch
Engineering Division

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

RICHARD DIBUONO, CHAIRMAN
Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and

rarity of such a storm event, finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

The Phase I investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespassing and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

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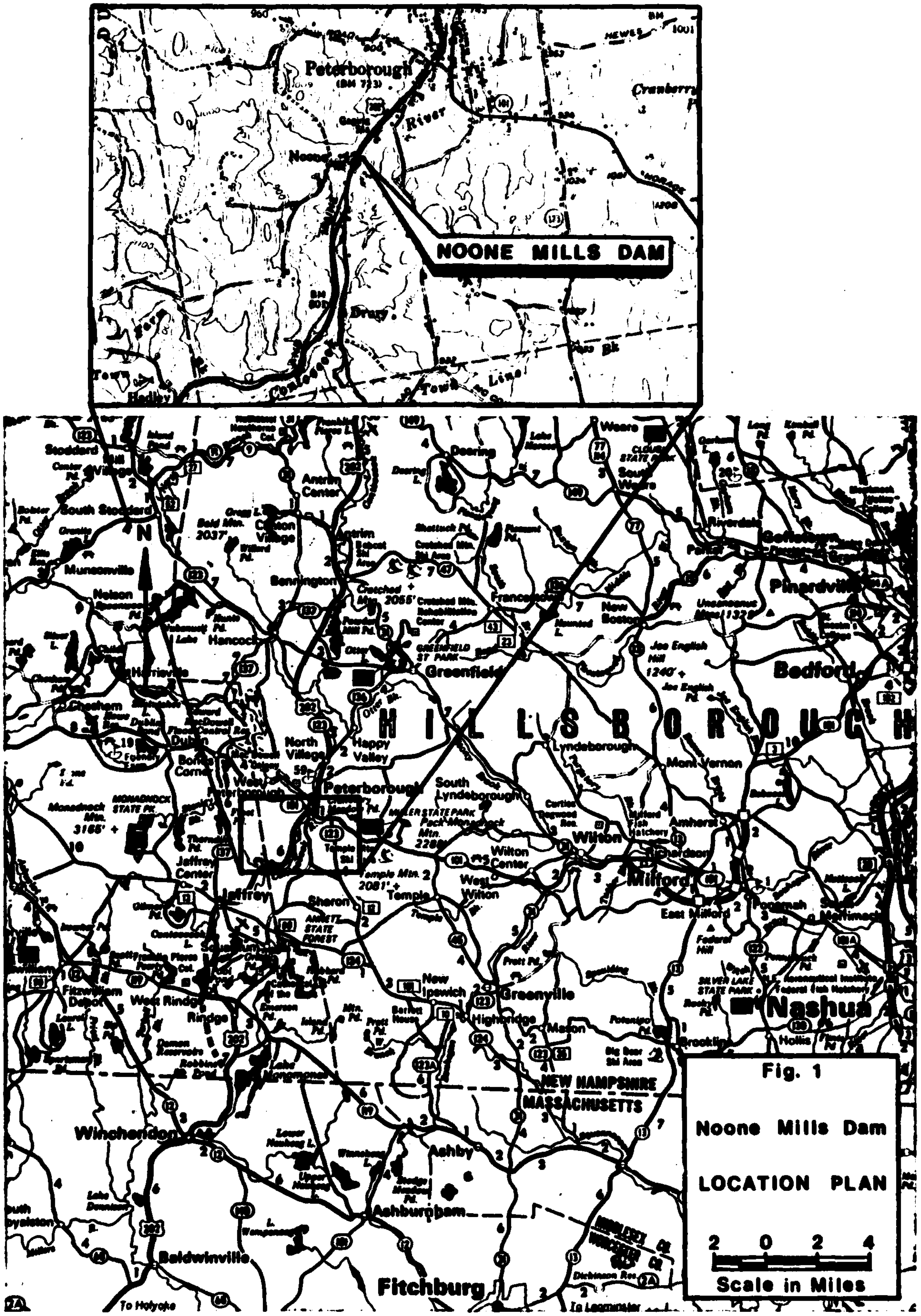
APPENDICES

OVERVIEW PHOTO - NOONE MILLS DAM


NOONE MILLS DAM
Peterborough
HILLSBOROUGH
NEW HAMPSHIRE
MASSACHUSETTS
Fig. 1
Noone Mills Dam
LOCATION PLAN
2 0 2 4
Scale in Miles

NATIONAL DAM INSPECTION PROGRAM
PHASE I INSPECTION REPORT
NOONE MILLS DAM

SECTION 1
PROJECT INFORMATION

1.1 General

a. Authority. Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. S E A Consultants Inc. has been retained by the New England Division to inspect and report on selected dams in the State of New Hampshire. Authorization and notice to proceed were issued to S E A Consultants Inc. under a letter of November 5, 1979 from William Hodgson, Jr., Colonel, Corps of Engineers. Contract No. DACW33-80-C-0008 has been assigned by the Corps of Engineers for this work.

b. Purpose

(1) To perform technical inspection and evaluation of non-federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-federal interests.

(2) To encourage and prepare the states to initiate quickly effective dam safety programs for non-federal dams.

(3) To update, verify and complete the National Inventory of Dams.

1.2 Description of Project

a. Location. Noone Mills Dam is located in the Town of Peterborough, New Hampshire, approximately 1.4 miles southwest from the center of town and just east of U.S. Route 202 at Noone Mills. The dam impounds water from the Contoocook River, which after passing over the spillway and overflow section continues to flow in a northeasterly direction through the center of Peterborough. The dam is shown on U.S.G.S. Quadrangle, Peterborough, New Hampshire, with coordinates approximately at N42°51'35", W71°57'44", Hillsborough County, New Hampshire. (See Location Plan.)

b. Description of Dam and Appurtenances. Noone Mills Dam is a stone filled, concrete capped gravity overflow structure, approximately 20 feet high from the top of the dam to the lowest ledge outcropping at the base and 267 feet long between abutments. The central portion of the dam consists of an overflow section

about 102 feet long between training walls. The upstream face consists of a concrete cap which extends from the top of the overflow section down approximately 1.7 feet to the top of a timber facing which slopes approximately 1 foot vertical to 1 foot horizontal (1:1) to the bottom of the river. The downstream slope of the overflow structure is also concrete capped with a batter of 1 foot vertical to 3.5 inches horizontal down to ledge outcropping at the base. The crest width is approximately 2 feet.

Located at the right abutment of the dam is the principal spillway which consists of a 34 feet long concrete spillway with flashboards. A 4.0 feet wide by 4.5 feet high sluice gate opening is located to the left of the spillway.

Located at the left abutment of the dam is the intake structure for a 5.5 feet diameter penstock which consists of a gate with lifting mechanism and a bar screen. A concrete retaining wall averaging 10 inches in thickness runs from the left abutment upstream approximately 277 feet and parallels the face of the mill buildings.

c. Size Classification. Small (height - 20 feet; storage - 315 acre-feet) based on storage (less than 1,000 acre-feet and greater than or equal to 50 acre-feet) as given in the Recommended Guidelines for Safety Inspection of Dams.

d. Hazard Classification. High Hazard. Failure of the Noone Mills Dam with water at the dam crest would cause an increase in stage of about 3 feet above the downstream tailwater or approximately 8 to 9 feet above the lower floor of the mill building located adjacent to the western side of the immediate downstream channel, and reach the unbricked portions of the mill building windows. In addition to this, the foundation of the building located immediately downstream from the left abutment of the dam could be undermined, causing extensive damage to this structure. Approximately 1 mile downstream, an increase of 2 feet in stage above the prefailure conditions would result in water reaching the sill level of a shopping center and light industrial buildings located just upstream from the New Hampshire Route 101 bridge. The potential for the loss of more than a few lives and extensive economic loss would exist.

e. Ownership. The initial structure was built in 1831, coinciding with the construction of the Noone Mills. Several large corporations have, at one time or another, owned Noone Mills as a subsidiary; the present organization being Darobsum Incorporated, 310 Marlboro Street, Keene, New Hampshire 03431, Telephone No. (603) 352-7587.

f. Operator. The dam is maintained and operated by Tim Brown, Manager, Darobsum Incorporated, Noone Mills Division, Peterborough, New Hampshire 03458, Telephone No. (603) 924-3091.

g. Purpose of Dam. The dam was originally built for industrial purposes. To date, water behind the dam is still being used by Noone Mills for processing.

h. Design and Construction History. Files at the State of New Hampshire Water Resources Board indicate the initial structure was built in 1831, coinciding with the construction of the Noone Mills. This was a stone dam with an upstream timber face. In 1923, the overflow section of the dam was capped with concrete

on the crest and downstream face. In 1936, the concrete retaining wall at the left abutment that parallels the mill buildings was constructed after a flood washed out this area. The "Flood of 1938" washed out a stone wing wall at the right abutment. It was replaced late in the same year with a concrete spillway cast on ledge and equipped with flashboards.

It is not known when the sluice gate at the right end of the overflow section was built, but photographs show it to be in existence by 1937. It is also not known when the penstock at the left abutment was built. Records indicate that power was generated as early as 1922, and the waterwheel and part of the penstock were dismantled and abandoned in 1961.

i. Normal Operating Procedures. The Noone Mills Dam is used primarily to retain the water of the Contoocook River for use in industrial processing. There is no normal operational procedure for this dam.

1.3 Pertinent Data

a. Drainage Area. The drainage area above the Noone Mills Dams covers approximately 71.0 square miles (45,440 acres), consisting of steeply sloped to moderately sloped terrain. The topography in the drainage basin ranges from 3,165 feet (NGVD) on top of Monadnock Mountain to 740 feet (NGVD) at the toe of the dam. A number of ponds and lakes are located in the upper portion of the drainage area. However, ponding areas are not nearly as evident in the lower portion of the basin, where numerous streams traverse the area and carry runoff to the Contoocook River. The drainage basin is predominantly tree covered and generally undeveloped. However, there are areas which do have significant development. These areas are concentrated in the town of Jaffrey and adjacent to the lakes in the upper portion of the drainage basin.

b. Discharge at Damsite. Discharge at the damsite normally occurs over the flashboards (set at Elev. 753.5 feet) which have been installed above the 34 feet long permanent spillway weir crest. Discharge will also occur over the 102 feet long overflow section when the pond water surface rises above elevation 754.0. A 4.5 feet high by 4.0 feet wide sluice gate is located in the dam face between the spillway and the overflow section. The gate was inoperable and leaking badly at the time of inspection. This gate would allow the pond to be lowered to an elevation of 745.4 feet. A 5.5 feet diameter penstock is located near the left abutment. The control mechanism for the penstock gate was not operable at the time of inspection.

(1) The capacity of the sluice gate was estimated to be 220 cfs with the water surface at the crest of the overflow section (Elev. 754.0 feet) and 310 cfs with the water surface at the top of dam (Elev. 760.3 feet).

(2) Maximum known flood at damsite - unknown

(3) The ungated spillway capacity with the water surface elevation at the top of the dam (elevation 760.3 feet) was estimated to be 2,200 cfs with the flashboards in place and 3,030 cfs with the flashboards removed.

(4) The ungated spillway capacity with the water surface elevation at the test flood elevation (Elev. 767.0 feet) was estimated to be 6,800 cfs with the flashboards in place and 6,980 cfs with the flashboards removed.

(5) N/A

(6) N/A

(7) The total capacity of the spillway (flashboards in place) and overflow section at the test flood elevation (Elev. 767.0 feet) was estimated to be 22,400 cfs (6,800 cfs spillway, 15,600 cfs overflow section).

(8) The total project discharge at the top of the dam (Elev. 760.3 feet) was estimated to be 8,600 cfs (2,200 cfs spillway; 5,200 cfs overflow section; 1,200 cfs various training walls and operator platform).

(9) The total project discharge at the test flood elevation (Elev. 767.0 feet) was estimated to be 37,600 cfs.

c. Elevation (feet NGVD) based on an elevation 754.0 extrapolated from U.S.G.S. quad sheet assumed to be pool elevation at top of overflow section.

(1) Streambed at toe of dam - 740.0

(2) Bottom of cutoff - unknown

(3) Maximum tailwater - unknown

(4) Normal pool - 754.0 (assumed to correspond with crest of overflow section)

(5) Full flood control pool - N/A

(6) Spillway crest (flashboards in place) 753.5
(flashboards removed) 751.3

(7) Design surcharge (Original Design) - unknown

(8) Top of dam - 760.3

(9) Test flood design surcharge - 767.0

d. Reservoir (length in feet)

(1) Normal pool - 1,650

(2) Flood control pool - N/A

(3) Spillway crest pool - 1,400 (flashboards in place)

(4) Top of dam - 3,350

(5) Test flood pool - 4,400

e. Storage (acre-feet)

(1) Normal pool - 135

(2) Flood control pool - N/A

(3) Spillway crest pool - 126 (flashboards in place)

(4) Top of dam - 315

(5) Test flood pool - 685

f. Reservoir Surface (acres)

(1) Normal pool - 19

(2) Flood control pool - N/A

(3) Spillway crest - 18.2 (flashboards in place)

(4) Test flood pool - 72

(5) Top of dam - 38.6

g. Dam

(1) Type - stone filled, concrete capped gravity overflow structure

(2) Length - 267 feet overall

(3) Height - 20 feet maximum

(4) Top Width - 2.8 feet

(5) Side Slopes - Upstream 1V to 1H timber facing to river bottom
Downsteam 1V to 0.29H concrete to river bottom

(6) Zoning - unknown

(7) Impervious core - not applicable

(8) Cutoff - unknown

(9) Grout curtain - unknown

h. Diversion and Regulating Tunnel

Not applicable (see Section j below)

i. Spillway

(1) Type - concrete with straight drop

(2) Length of weir - 34.0 feet

(3) Crest elevation - 753.5 (with flashboards)
751.3 (permanent crest)

(4) Gates - N/A

(5) U/S Channel - A bridge is located approximately 370 feet upstream from the spillway. A concrete wall extends from the bridge to the dam on the western (left) side of the channel. The river face of this concrete is severely eroded. The eastern (right) side of the channel is tree-lined with some trees overhanging the channel. The slopes appear to be stable.

(6) D/S Channel. The spillway discharges into a natural river channel, which is separate from the natural stream channel that the overflow section discharges to. Approximately 300 to 400 feet below the spillway, these two channels join, creating a single channel which is about 50 feet wide. The channel flows in a northeasterly direction passing beneath a town road and then NH Route 101 (approximately 1 mile downstream) before flowing through the downtown area of Peterborough.

j. Regulating Outlets

(1) Invert - Sluice gate - 745.4 bottom of discharge opening

(2) Size - Sluice gate - 4.0 feet wide by 4.5 feet high opening

(3) Description - Sluice gate - one wooden gate with 4.0 feet by 4.5 feet opening

(4) Control Mechanism - Sluice gate - Manual handwheel type operator (inoperable at time of inspection)

(5) Other - Intake for 5.5 diameter penstock controlled by manual handwheel type operator (inoperable at time of inspection). Penstock and works abandoned.

Process water pipe - Apparently, a process water pipe intake is located just upstream from the penstock intake structure. The present owner does not know the pipe size or material. The pipe could not be seen during the inspection since it was submerged.

SECTION 2
ENGINEERING DATA

2.1 Design

No design data were disclosed for Noone Mills Dam.

2.2 Construction

Records from the State of New Hampshire Water Resources Board, indicate the dam was reconstructed and repaired in 1938. A memorandum dated November 15, 1938 briefly outlines the reconstruction and repairs made to the dam by the Iafolla Construction Company of Portsmouth, New Hamphsire. A sketch dated April 24, 1939 shows as-built detail of the dam.

2.3 Operation

No engineering operational data were disclosed.

2.4 Evaluation

a. Availability. No engineering data were available for Noone Mills Dam, other than a construction memorandum and an "as-built" sketch described in Section 2.2. A search of the files of the State of New Hampshire Water Resources Board and contact with the manager of Noone Mills, revealed a limited amount of recorded information.

b. Adequacy. The final assessments and recommendations of this investigation are based on the visual inspection and the hydrologic and hydraulic calculations.

c. Validity. The field investigation indicated that the external features of the Noone Mills Dam generally agree with those shown on the "as-built" sketch mentioned in Section 2.2, although the dimensions given in this section are totally inaccurate.

SECTION 3
VISUAL INSPECTION

3.1 Findings

a. General. Noone Mills Dam impounds a pond area of small size. The drainage area above the dam consists of a steeply sloped to moderately sloped terrain. The majority of the basin is predominantly tree covered and generally undeveloped except for areas in the town of Jaffrey and adjacent to the lakes in the upper portion of the drainage basin. The downstream area is generally undeveloped until the channel reaches the town of Peterborough.

The field inspection of Noone Mills Dam was made on December 11, 1979. The inspection team consisted of personnel from S E A Consultants Inc. and Geotechnical Engineers, Inc. Inspection checklists completed during the visual inspection are included in Appendix A. At the time of inspection, water was passing approximately 2 inches deep over the 102 feet wide overflow section and approximately 5 inches deep over the 34 feet wide spillway. The pool elevation was at approximately 754.3 NGVD. The upstream face of the dam could only be inspected above this water level. Inspection of the downstream face of the overflow section and spillway was not possible due to the discharge of water.

b. Dam. Noone Mills Dam is a stone filled, concrete capped, gravity overflow structure, approximately 20 feet high from the top of the dam to the lowest ledge outcropping at the base, and 267 feet long between abutments.

The central portion of the dam consists of a stone filled, concrete capped, overflow section about 102 feet long between training walls. (See Photo Nos. 2, 5, and 18.) The upstream face consists of a concrete cap which extends from the top of the overflow section down approximately 1.7 feet to the top of a timber facing which slopes approximately 1 foot vertical to 1 foot horizontal (1:1) to the bottom of the river. (See Plans and Details in Appendix B.) The downstream slope of the overflow section is also concrete capped with a batter of 1 foot vertical to 3.5 inches horizontal down to ledge outcropping at the base. The crest width is approximately 2 feet. The left training wall is constructed of concrete and is severely eroded and spalled on the first 2 feet in from the overflow section. (See Photo No. 6.) The right training wall is also constructed of concrete and acts as a buttress for the concrete overflow section. (See Photo No. 8.) The concrete is severely spalled on the downstream face of this buttress, and there is a one cubic yard cavity in the concrete at the toe of the buttress exposing the reinforcing steel. (See Photo Nos. 9 and 10.) Located to the right of center in the overflow section are two stone filled, concrete capped buttresses on the downstream face. There is severe erosion of the concrete cap on both of these buttresses exposing the rock fill. (See Photo No. 10.)

c. Appurtenant Structures. Located near the left abutment is a concrete embankment wall which extends from the left training wall of the overflow section to the mill buildings at the left abutment. (See Plans and Details in Appendix B.) The downstream face of this wall has the same batter and is in the same line of plane as the overflow section. The top of this wall is at the same height as the

training walls. The downstream face is severely spalled in several locations, and there are at least two locations where vegetation is growing out of cracks in the wall, with visible seepage. There is also some vegetation growth on top of the wall near the mill buildings. A moderate amount of seepage was observed through the toe of the wall near the left training wall.

The penstock is located through this concrete embankment wall and consists of a concrete intake structure with gate lifting mechanism and bar screening. (See Photo No. 4.) A 5.5 feet diameter riveted steel plate penstock extends from the intake structure through the concrete embankment wall and continues downstream for about 100 feet where it terminates. The penstock has been abandoned since 1961 and many sections have been removed. The penstock gate is closed and inoperable, the lifting mechanism is severely rusted, and wood members of the gate mechanism have rotted. There is some minor leakage through the gate which eventually discharges into the river through large holes in the bottom of the penstock. The concrete intake structure for the penstock is spalled considerably and the bar screening is covered with rust.

Beginning at the penstock intake structure is a concrete retaining wall averaging 10 inches in thickness which runs from the intake structure upstream approximately 277 feet and parallels the face of the mill buildings. (See Photo Nos. 3 and 4) This wall extends along the left side of the penstock intake structure, intersects the dam embankment wall approximately 25 feet from the left abutment, and runs on top of the dam embankment wall, a distance of approximately 32 feet where it terminates at the left training wall of the overflow section. The wall has an average height of about 6.5 feet above water surface. The river face is severely eroded at the water surface, probably due to ice damage. (See Photo Nos. 3 and 4.) There are large vertical cracks through the wall at several locations. The ground surface behind the wall is about 2 feet below the top of the wall and is flat and sparsely vegetated. There is one large pine tree and a considerable amount of brush growing immediately behind this wall. (See Photo No. 3.)

Located to the right of the overflow section is a concrete embankment wall which extends from the right training wall of the overflow section to the spillway. The downstream face of this wall has the same batter and is in the same line of plane as the overflow section. The top of this wall is at the same height as the training walls. The downstream face is severely spalled at lower elevations, and there is some slight seepage through the wall where it joins the buttress that forms the right training wall of the overflow section. (See Photo No. 13.) A small tree is growing out of the toe of the wall. (See Photo No. 13.)

The dam's low level outlet structure is located through this concrete embankment wall and consists of a 4.0 foot wide by 4.5 foot high sluice gate opening with a wooden gate and handwheel operated lifting mechanism. (See Photo No. 11.) The gate is closed and inoperable, the lifting mechanism is severely rusted, and the wooden gate stems have decayed and rotted. There is severe leakage through the sluice gate, and the concrete is severely eroded and deteriorated at the gate opening. (See Photo No. 12.) Several trees, up to 6 inches in diameter, are growing near the base of the gate structure at the outlet.

The principal spillway is located at the right abutment of the dam. (See Photo Nos. 2 and 14.) It is a concrete spillway structure with a total length of approximately 55 feet from the concrete embankment wall on the left side of the spillway to the right abutment. The weir length is about 34 feet, and the permanent crest can accomodate removable flashboards to raise the ponding elevation approximately 2 feet. (See Photo No. 16.) A walkway consisting of two 10-inch deep steel beams and wood planking spans the spillway to give access to the sluice gate operator. (See Photo Nos. 15 and 16.) Several of these planks are either missing or rotted. The steel beams are covered with rust, and the right concrete abutment of the walkway is spalled and rust stained. The flashboards appear to be in good shape, but some debris has collected behind the flashboards at the spillway crest. (See Photo Nos. 15 and 16.)

The right abutment is thickly vegetated with brush and small trees. (See Photo No. 15.) Occasional trees up to 6 inches in diameter are present. Irregular riprap covers some of the abutment slope. This riprap appears to be stable.

d. Reservoir Area. The slopes of the reservoir appear stable. Sedimentation behind the dam and spillway was not observable. There are several small overhanging trees and one fallen tree along the approach channel to the spillway. (See Photo No. 14.) The channel is wide and otherwise unobstructed.

e. Downstream Channel. The spillway discharge channel is cluttered with occasional fallen limbs, rock blocks, and one large concrete block. Both sides of the channel are lined with trees, with occasional trees overhanging the channel floor. (See Photo No. 17.)

A few small trees are growing out of the channel floor downstream of the dam overflow section. Occasional fallen limbs and logs are in the discharge channel. Both sides of the channel are tree-lined with occasional trees overhanging the channel floor.

3.2 Evaluation

On the basis of the results of the visual inspection, Noone Mills Dam is considered to be in poor condition.

A one cubic yard cavity in the concrete exposing the reinforcing steel at the base of the buttress forming the right training wall of the overflow section, the severe spalling of the downstream face of this buttress, and the severe erosion of the concrete capped buttresses on the downstream face of the overflow section exposing the rock fill are signs of serious structural problems to the overflow section. It is possible that the overflow section could fail at any time.

The severe erosion at the water surface of the 10 inch thick concrete retaining wall that parallels the mill buildings, and several large vertical cracks in this wall are signs of serious structural and stability problems. This wall may eventually collapse, which would probably not affect the stability of the dam, but may affect the integrity of the foundations of the mill building on this side of the reservoir.

Leakage through cracks in the left and right embankment walls, major seepage at the toe of the left embankment wall near the left training wall, and severe spalling and deterioration of the concrete on the downstream face of these walls are all signs of severe structural problems. Continued seepage could accelerate damage and deterioration of these walls and eventually lead to failure of the walls.

Trees and vegetation growing in or near cracks in the concrete at several locations on the dam could cause further propagation of these cracks and further deterioration of the concrete.

A severe leak in the sluice gate with a large amount of discharge and the inoperability of the gate lifting mechanism are signs of considerable deterioration of the gate. It is possible the gate could fail at any time.

A leak in the penstock gate, the inoperability of the gate lifting mechanism, and considerable spalling of the concrete intake structure are signs of considerable deterioration of the gate. If the leakage is not corrected, it could lead to further deterioration and eventual failure of the gate.

Trees growing on the banks of the river near the dam on the upstream and downstream side and brush which will eventually attain tree-size behind the retaining wall that parallels the mill buildings may lead to erosion and seepage problems if a tree blows over and pulls out its roots, or if a tree dies or is cut and its roots rot.

SECTION 4
OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 Operational Procedures

a. General. The Noone Mills Dam is used primarily to impound water from the Contoocook River. There are no written or routine operational procedures.

b. Description of Any Warning System in Effect. No written warning system exists for the dam.

4.2 Maintenance Procedures

a. General. The owner, Darobsun Incorporated, Noone Mills Division, Tim Brown, manager, is responsible for the maintenance of the dam. No formal maintenance was discussed.

b. Operating Facilities. No formal plan for maintenance of operating facilities was disclosed.

4.3 Evaluation

The current maintenance procedures for Noone Mills Dam are inadequate to insure that all problems encountered can be remedied within a reasonable period of time. The owner should establish a written operation and maintenance procedure, as well as establish a warning system to follow in event of flood flow conditions or imminent dam failure.

SECTION 5
EVALUATION OF HYDROLOGIC/HYDRAULIC FEATURES

5.1 General. The Noone Mills Dam is a stone filled, concrete capped gravity overflow structure, approximately 20 feet high from top of dam to the lowest ledge outcropping at the base and 267 feet long between abutments. The overflow section is approximately 102 feet long and set at an elevation of 754.0 feet. Located at the right abutment of the dam is the principal spillway consisting of a permanent concrete spillway crest with flashboards installed to an elevation of 753.5 feet. A 4.0 feet wide by 4.5 feet high sluice gate opening is located to the left of the spillway. The sluice gate is inoperable and leaking badly.

Numerous ponds and lakes are located in the upper (southwestern) portion of the drainage basin. Consequently, storm water deposited in this portion of the basin would be intercepted by these water bodies before being discharged to the Contoocook River and flowing to the Noone Mills Dam. These lakes and ponds are not evident in the lower (northeastern) portion of the basin. Runoff from this portion of the basin would be passed more quickly to the Contoocook River, by streams traversing the area. The Noone Mills Dam is a run of the river type project, having a maximum storage of 315 acre-feet.

5.2 Design Data. No hydrological or hydraulic design data were disclosed.

5.3 Experience Data. No experience data were disclosed. Maximum flood flows or elevations are unknown. However, it is known that the "Flood of 1938" washed out a stone wing wall at the right abutment, which was replaced late the same year with the concrete spillway.

5.4 Test Flood Analysis. Due to the absence of detailed design and operational information, the hydrologic evaluation was performed utilizing data gathered during field inspection, watershed size and an estimated test flood equal to one-half the Probable Maximum Flood (1/2 PMF) as determined with the "rolling" curve from the Corps of Engineers guide curves. For a dam of small size and high hazard the test flood ranges from one-half the Probable Maximum Flood (1/2 PMF) to the full Probable Maximum Flood (PMF). The 1/2 PMF was selected for this analysis since the Noone Mills Dam falls on the lower end of the small size range. The rolling curve was used to estimate the maximum probable flood peak flow rate since it is representative of the type of topography found in much of the drainage basin, and also since it represents a balance between the mountainous portions of the basin and the areas with numerous lakes and ponds.

Based on a maximum probable flood peak flow rate of 1,075 cfs per square mile and on a drainage area of 71 square miles, the test flood inflow was estimated to be 38,200 cfs. The test flood was routed through the reservoir in accordance with the Corps of Engineers procedure for Estimating Effect of Surcharge Storage on Maximum Probable Discharge. The pond water surface was assumed to be at Elev. 754.0 prior to the flood routing. The routed test flood outflow was estimated

to be 37,600 cfs. This analysis indicated that the dam crest would be overtopped by 6.7 feet. The maximum spillway capacity (assuming that the sluice gate is closed and the flashboards are in place) with the water level at the dam crest was estimated to be 2,200 cfs, which is only about 6 percent of the routed test flood outflow. The capacity of the spillway and overflow section combined with the water level at the dam crest was estimated to be 7,400 cfs, which is only about 20 percent of the routed test flood outflow.

5.5 Dam Failure Analysis. The impact of dam failure was assessed utilizing the "Rule of Thumb" Guidance for Estimating Downstream Dam Failure Hydrographs published by the Corps of Engineers. The analysis covered a reach extending approximately 1.4 miles downstream. The prefailure discharge with the water surface at the dam crest is significant, so prefailure tailwater conditions were included in the hydrologic calculations and the dam failure analysis was conducted with the water surface at the dam crest. Under these conditions, it was determined that the routed dam failure discharge would significantly increase the hazard over the prefailure discharge tailwater. Based on this analysis, Noone Mills Dam has been classified as a high hazard.

Due to the general condition of the stone filled, concrete capped overflow section, it was determined that this section of the dam represented the most probable place for an assumed breach to occur. Consequently, a total of 102 feet of the dam, equal to the length of the overflow section, was breached with a failure height of about 20 feet. The total failure discharge was estimated to be 19,100 cfs, which included a discharge of 15,700 cfs through the breached section plus discharge over the unfailed portion of the spillway. The spillway discharge immediately prior to failure was estimated to be 8,600 cfs.

An assumed failure of the Noone Mills Dam with the water surface at the dam crest (Elev. 760.3 feet) would increase the stage along the immediate downstream channel to about 12 feet, more than 3 feet above the stage of the prefailure tailwater. This would result in a water depth of approximately 8 to 9 feet above the sill of the lower floor of the mill building located adjacent to the western side of the downstream channel, and reach the unbricked portions of the mill building windows. In addition to this, the foundation of the building located immediately downstream from the left abutment of the dam could be undermined, causing extensive damage to this structure. Approximately one-half mile downstream, the channel cross-section broadens and the stage would be reduced to just over 6 feet which is only about a one foot increase in stage above prefailure tailwater. However, as the channel approaches the Route 101 bridge, approximately 1 mile downstream, it has a lesser slope and more steeply sloped banks, which present a smaller cross-section available to accommodate the discharge. Consequently, the stage in this stretch of the river would increase to more than 12 feet in order to pass the discharge, which is more than 2 feet above the stage of the prefailure tailwater. The prefailure flow would reach the parking lot and approach the sill of the shopping center buildings. The increase in stage resulting from the failure discharge would cause water to rise to above the sill of the shopping center buildings and impact light industrial buildings located near the shopping center. The potential for the loss of more than a few lives and extensive economic loss would exist.

SECTION 6
EVALUATION OF STRUCTURAL STABILITY

6.1 Visual Observations

The visual inspection indicates the following potential structural problems:

(1) A one cubic yard cavity in the concrete exposing the reinforcing steel at the base of the buttress forming the right training wall of the overflow section, the severe spalling of the downstream face of this buttress, and the severe erosion of the concrete capped buttresses on the downstream face of the overflow section exposing the rock fill are signs of serious structural problems to the overflow section. It is possible that the overflow section could fail at any time.

(2) The severe erosion at the water surface of the 10 inch thick concrete retaining wall that parallels the mill buildings and several large vertical cracks in this wall are signs of serious structural and stability problems. This wall may eventually collapse, which would probably not affect the stability of the dam, but may affect the integrity of the foundations of the mill building on this side of the reservoir.

(3) Leakage through cracks in the left and right embankment walls, major seepage at the toe of the left embankment wall near the left training wall, and severe spalling and deterioration of the concrete on the downstream face of these walls are all signs of severe structural problems. Continued seepage could accelerate damage and deterioration of these walls and eventually lead to failure of the walls.

(4) Trees and vegetation growing in or near cracks in the concrete at several locations on the dam with visible seepage could cause further propagation of these cracks and further deterioration of the concrete.

(5) The severe leak in the sluice gate with a large amount of discharge and the inoperability of the gate lifting mechanism are signs of considerable deterioration of the gate. It is possible that the gate could fail at any time.

(6) The leak in the penstock gate, the inoperability of the gate lifting mechanism, and the considerable spalling of the concrete intake structure are signs of considerable deterioration of the gate. If the leakage is not corrected, it could lead to further deterioration and eventual failure of the gate.

(7) Trees growing on the banks of the river near the dam on the upstream and downstream side and brush which will eventually attain tree-size behind the retaining wall that parallels the mill buildings may lead to erosion and seepage problems if a tree blows over and pulls out its roots, or if a tree dies or is cut and its roots rot.

6.2 Design and Construction Data

No information regarding the original design or construction of the dam was found, although it is known that the early structures of the dam were built in 1831, coinciding with the construction of Noone Mills.

6.3 Post-Construction Changes

In 1923, the overflow section of the original stone dam was capped with concrete on the crest and downstream face. In 1936, the concrete retaining wall at the left abutment that parallels the mill buildings was constructed after a flood washed out this area. The flood of 1938 washed out a stone wing wall at the right abutment. It was replaced late in the same year with the present concrete spillway cast on ledge.

It is not known when the sluice gate at the right end of the overflow section was built, but photographs show it to be in existence by 1937. It is also not known when the penstock at the left abutment was built. Records indicate that power was generated as early as 1922, and the waterwheel and part of the penstock were dismantled and abandoned in 1961.

6.4 Seismic Stability

This dam is located in Seismic Zone 2 and, in accordance with the Phase I guidelines, does not warrant seismic analysis.

SECTION 7
ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition. The visual examination indicates that Noone Mills Dam is in poor condition. The major concerns with respect to the integrity of the dam are:

(1) The one cubic yard cavity in the concrete exposing the reinforcing steel at the base of the buttress forming the right training wall of the overflow section and the severe spalling of the downstream face of this buttress.

(2) The severe erosion of the concrete capped buttresses on the downstream face of the overflow section exposing the rock fill.

(3) The severe erosion at the water surface of the 10 inch thick concrete retaining wall that parallels the mill buildings and the several large vertical cracks in this wall.

(4) Leakage through cracks in the left embankment wall, major seepage at the toe of the left embankment wall near the left training wall, and the severe spalling and deterioration of the concrete on the downstream face of this wall.

(5) Leakage through a crack in the right embankment wall and the severe spalling and deterioration of the concrete on the downstream face of this wall at the lower elevations.

(6) Trees and vegetation growing in or near cracks in the concrete at several locations on the dam.

(7) The severe leak in the sluice gate and the inoperability of the gate lifting mechanism.

(8) The leak in the penstock gate, the inoperability of the gate lifting mechanism, and the considerable spalling of the concrete intake structure.

(9) Trees growing on the banks of the river near the dam on the upstream and downstream side and brush which will eventually attain tree-size behind the retaining wall that parallels the mill buildings.

(10) Inadequacy of the spillway and overflow section to pass the test flood.

b. Adequacy of Information. The information available from the visual inspection is adequate to identify the problems that are listed in 7.2. These problems will require the attention of a qualified registered professional engineer who will have to make additional engineering studies to design or specify remedial measures. No additional information is needed for the purpose of this Phase I investigation.

c. Urgency. The owner should implement the recommendations in 7.2 and 7.3 within one year after receipt of this Phase I report.

7.2 Recommendations. The reservoir should be lowered enough to permit an inspection of the dam and spillway masonry and concrete structures.

The owner should retain a registered professional engineer qualified in the design and construction of dams to:

(1) Investigate the one cubic yard cavity in the concrete exposing the reinforcing steel at the base of the buttress forming the right training wall of the overflow section and the severe spalling of the downstream face of this buttress, and design remedial meaures as needed.

(2) Investigate the severe erosion of the concrete capped buttresses on the downstream face of the overflow section exposing the rock fill and design remedial measures as needed.

(3) Investigate the severe erosion at the water surface of the 10 inch thick concrete retaining wall that parallels the mill buildings and the several large vertical cracks in this wall, and design remedial measures as needed.

(4) Investigate the leakage through cracks in the left embankment wall, major seepage at the toe of the left embankment wall near the left training wall, and the severe spalling and deterioration of the concrete on the downstream face of this wall, and design remedial measures as needed.

(5) Investigate the leakage through a crack in the right embankment wall and the severe spalling and deterioration of the concrete on the downstream face of this wall at the lower elevations, and design remedial measures as needed.

(6) Investigate the severe leak in the sluice gate and the inoperability of the gate lifting mechanism, and design remedial measures as needed.

(7) Investigate the leak in the penstock gate, the inoperability of the gate lifting mechanism, and the considerable spalling of the concrete intake structure, and design remedial measures as needed.

(8) Inspect the dam and spillway and the 10-inch concrete retaining wall on the left side of the reservoir that parallels the mill buildings when the reservoir is lowered.

(9) Do a detailed hydrologic-hydraulic investigation to assess further the potential for overtopping the dam, the adequacy of the spillway to pass the test flood, and the need for and means to increase project discharge capacity.

The owner should carry out the recommendations made by the engineer.

7.3 Remedial Measures

a. Operating and Maintenance Procedures. The owner should:

(1) Remove overhanging trees near the approach channel to the spillway and at the discharge channel downstream of both the overflow section and the spillway.

(2) Remove trees near the base of the sluice gate outlet structure.

(3) Remove the debris and trees at the spillway crest and in the discharge channels.

(4) Remove the large pine tree and brush growing behind the 10-inch retaining wall that parallels the mill building.

(5) Establish a regular operation and maintenance program.

(6) Engage a registered professional engineer qualified in the design and construction of dams to make a comprehensive technical inspection of the dam once every year after the recommendations made in 7.2 have been carried out.

(7) Establish a surveillance program for use during and immediately after heavy rainfall, and also a warning program to follow in case of emergency conditions.

7.4 Alternatives

There are no practical alternatives to the recommendations of Section 7.2 and 7.3.

APPENDIX A

INSPECTION CHECK LIST

INSPECTION CHECK LIST
PARTY ORGANIZATION

PROJECT: Noone Mills Dam, NH

DATE: December 11, 1979

TIME: 1:00 P.M.

WEATHER: Sunny, warm

W.S. ELEV. 754.4 U.S. 742.3 DN.S.
(NGVD)

PARTY:

1. Robert Durfee, S E A
2. Bruce Pierstorff, S E A
3. Philip Ricardi, S E A
4. Philip Upton, S E A
5. Karl Dalenberg, GEI
6. Kenneth Stern, NHWRB
7. Richard DeBold, NHWRB
8. Ronald Hirschfeld, (GEI) 2/29/80
9.
10.

	PROJECT FEATURE	INSPECTED BY	REMARKS
1.	Structural stability	R. Durfee/P. Upton	
2.	Hydrology/hydraulics	B. Pierstorff/P. Ricardi	
3.	Soils and geology	R. Hirschfeld/K. Dalenberg	
4.			
5.			
6.			
7.			
8.			
9.			
10.			

INSPECTION CHECK LIST

PROJECT: Noone Mills Dam, NH — DATE: December 11, 1979

PROJECT FEATURE: Dam Embankment — NAME:

DISCIPLINE: — NAME:

AREA EVALUATED	CONDITIONS
DAM EMBANKMENT	
Crest Elevation	754.0
Current Pool Elevation	754.3
Maximum Impoundment to Date	Unknown
Surface Cracks	Numerous throughout structure
Pavement Condition	No pavement
Movement or Settlement of Crest	None observed
Lateral Movement	None observed
Vertical Alignment	Good
Horizontal Alignment	Good
Condition At Abutment and at Concrete Structures	Poor - concrete severely deteriorated at numerous locations
Indications of Movement of Structural Items on Slopes	None observed
Trespassing on Slopes	None observed
Vegetation on Slopes	Numerous locations of vegetation growing out of cracks in concrete
Sloughing or Erosion of Slopes or Abutments	None observed
Rock Slope Protection - Riprap Failures	Not applicable
Unusual Movement or Cracking at or near Toe	Several locations of cracking at downstream toe
Unusual Embankment or Downstream Seepage	Several locations of seepage through cracks in concrete
Piping or Boils	None observed
Foundation Drainage Features	None
Toe Drains	None
Instrumentation System	None

INSPECTION CHECK LIST

PROJECT: Noone Mills Dam, NH

DATE: December 11, 1979

PROJECT FEATURE: Dike Embankment

NAME:

DISCIPLINE:

NAME:

AREA EVALUATED	CONDITIONS
DIKE EMBANKMENT	No dike
Crest Elevation	
Current Pool Elevation	
Maximum Impoundment to Date	
Surface Cracks	
Pavement Condition	
Movement or Settlement of Crest	
Lateral Movement	
Vertical Alignment	
Horizontal Alignment	
Condition at Abutment and at Concrete Structures	
Indications of Movement of Structural Items on Slopes	
Trespassing on Slopes	
Vegetation on Slopes	
Sloughing or Erosion of Slopes or Abutments	
Rock Slope Protection - Riprap Failures	
Unusual Movement or Cracking at or near Toes	
Unusual Embankment or Downstream Seepage	
Piping or Boils	
Foundation Drainage Features	
Toe Drains	
Instrumentation System	

INSPECTION CHECK LIST

PROJECT: Noone Mills Dam, NH DATE: December 11, 1979

PROJECT FEATURE: Intake Channel NAME:

DISCIPLINE: NAME:

AREA EVALUATED	CONDITIONS: SLUICE GATE	CONDITIONS: PENSTOCK
OUTLET WORKS - INTAKE CHANNEL AND INTAKE STRUCTURE	4.0 x 4.5 foot sluice gate opening; gate closed and abandoned	5.5 foot diameter Penstock; Penstock gate closed and abandoned
a. Approach Channel		
Slope Conditions	No slopes observable	Left side of channel is concrete retaining wall
Bottom Conditions	Under water, not observable	Under water, not observable
Rock Slides or Falls	None observable	None observable
Log Boom	None	Bar screening heavily rusted
Debris	None observed	None observed
Condition of Concrete Lining	Under water, not observable	Under water, not observable
Drains or Weep Holes	None observed	None observed
b. Intake Structure		
Condition of Concrete	Under water, not observable	Considerable spalling above water surface elevation
Stop Logs and Slots	None	None

INSPECTION CHECK LIST

PROJECT: Noone Mills Dam, NH DATE: December 11, 1979

PROJECT FEATURE: Control Tower NAME:

DISCIPLINE: NAME:

AREA EVALUATED	CONDITIONS: SLUICE GATE	CONDITIONS: PENSTOCK
OUTLET WORKS - CONTROL TOWER	Control works located on top of concrete dam embankment	Control works located on top of penstock intake structure
a. Concrete and Structural		
General Condition	Very poor	Very poor
Condition of Joints	None	None
Spalling	Minor spalling at top of embankment wall	Several locations of severe spalling
Visible Reinforcing	None	None
Rusting or Staining of Concrete	Staining of concrete below operating mechanism	Staining of concrete below operating mechanism
Any Seepage or Efflorescence	None observed	None observed
Joint Alignment	Good	Good
Unusual Seepage or Leaks in Gate Chamber	Severe leak through sluice gate	Minor leak through penstock gate
Cracks	Minor	Minor
Rusting or Corrosion of Steel	Operating mechanism heavily rusted and inoperable	Operating mechanism heavily rusted and inoperable
b. Mechanical and Electrical		
Air Vents	None	Vent through penstock conduit
Float Wells	None	None
Crane Hoist	None	None
Elevator	None	None
Hydraulic System	None	None
Service Gates	Closed and inoperable; severe leakage	Closed and inoperable; minor leakage
Emergency Gates	See service gates	See service gates
Lightning Protection System	None	None
Emergency Power System	None	None
Wiring and Lighting System in Gate Chamber	None	None

INSPECTION CHECK LIST

PROJECT: Noone Mills Dam, NH

DATE: December 11, 1979

PROJECT FEATURE: Transition and conduit

NAME:

DISCIPLINE:

NAME:

AREA EVALUATED	CONDITIONS: SLUICE GATE	CONDITIONS: PENSTOCK
OUTLET WORKS - TRANSITION AND CONDUIT	No transition and conduit	5.5 foot diameter riveted steel plate Penstock conduit
General Condition of Concrete		
Rust or Staining on Concrete		Severe rusting of 5.5 foot diameter conduit
Spalling		Not applicable
Erosion or Cavitation		Bottom of Penstock conduit eroded away at several locations
Cracking		Not applicable
Alignment of Monoliths		Not applicable
Alignment of Joints		Several joints split open
Numbering of Monoliths		Not applicable

INSPECTION CHECK LIST

PROJECT: Noone Mills Dam, NH DATE: December 11, 1979

PROJECT FEATURE: Outlet Structure NAME:

DISCIPLINE: NAME:

AREA EVALUATED	CONDITIONS SLUICE GATE	PENSTOCK
OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL		Penstock abandoned. Several sections of conduit removed. No outlet provided
General Condition of Concrete	Very poor on down-stream face of concrete.	
Rust or Staining	None observed	
Spalling	Severe spalling at lower elevations	
Erosion or Cavitation	Severe erosion at lower elevation	
Visible Reinforcing	None	
Any Seepage or Efflorescence	Severe leakage through gate	
Condition at Joints	Not applicable	
Drain holes	None	
Channel	Bottom of channel is bedrock.	
Loose Rock or Trees Overhanging Channel	Several trees up to 5-6 in. and occasional loose blocks on right side.	
Condition of Discharge Channel	Some loose blocks at junction of channel with main discharge channel	

INSPECTION CHECK LIST

PROJECT: Noone Mills Dam, NH DATE: December 11, 1979

PROJECT FEATURE: Spillway Weir NAME:

DISCIPLINE: NAME:

AREA EVALUATED	CONDITIONS SPILLWAY	OVERFLOW SECTION
OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS		
a. Approach Channel		
General Conditions	Some debris hung up on stoplogs at center. Channel below water, not observable.	Below water, not observable
Loose Rock Overhanging Channel	Rocks on right side appear stable	None
Trees Overhanging Channel	Several trees up to 4-5 in. on right side of channel	None
Floor of Approach Channel	Under water, not observable	Under water, not observable
b. Weir and Training Walls		
General Condition of Concrete	Good	Very poor
Rust or Staining	None observed	None
Spalling	None observed	Severe throughout overflow section
Any Visible Reinforcing	None observed	Large cavity in concrete exposing rebar at toe of right training wall
Any Seepage of Efflorescence	None observed	Severe leaks throughout overflow section.
Drain Holes	Not observable	Not observable
c. Discharge channel		
General Condition	Fair, several obstacles present	Same
Loose Rock Overhanging Channel	Some loose blocks on right abutment just downstream of crest	None
Trees Overhanging Channel	Trees line both sides of channel beginning at edge of water	Same
Floor of Channel	Bedrock bottom	Same
Other Obstructions	Loose blocks and one large mass of concrete 4'x8'x2' in bottom of channel.	Some loose blocks, 1-2 in. trees in channel floor.

INSPECTION CHECK LIST

PROJECT: Noone Mills Dam, NH DATE: December 11, 1979

PROJECT FEATURE: Service Bridge NAME:

DISCIPLINE: NAME:

AREA EVALUATED	CONDITIONS
<u>OUTLET WORKS - SERVICE BRIDGE</u>	Service bridge (walkway) located over spill-way weir
a. Super Structure	
Bearings	Longitudinal members embedded in concrete
Anchor Bolts	No anchor bolts. Longitudinal members embedded in concrete.
Bridge Seat	Spalling of concrete embedment around longitudinal members at right end of walkway
Longitudinal Members	Two 10-inch deep steel I beams, covered with rust
Under Side of Deck	Longitudinal members rusted
Secondary Bracing	None
Deck	Wood planks, several rotted and missing
Drainage System	None
Railings	Railing rusted and wobbly
Expansion Joints	None
Paint	No paint. All steel members covered with rust.
b. Abutment & Piers	
General Condition of Concrete	Good
Alignment of Abutment	Good
Approach to Bridge	Footpath, clear and open
Condition of Seat & Backwall	Good

AVAILABLE ENGINEERING DATA

A sketch dated 1939 showing "as-built" detail of plan and section for reconstruction of the Noone Mills Dam is available at the New Hampshire Water Resources Board, 37 Pleasant Street, Concord, New Hampshire 03301.

It should be noted that the field investigation indicated that the external features of the Noone Mills Dam generally agree with those shown on the "as-built" sketch, although the dimensions given in this sketch are totally inaccurate.

PAST INSPECTION REPORTS

State of New Hampshire
WATER RESOURCES BOARD
37 Pleasant Street
Concord, N.H. 03301
TELEPHONE 271-3406

December 20, 1979

Mr. Timothy Brown, Manager
Noone Mills
Peterborough, New Hampshire 03458

Subject: Dam No. 191.02

Dear Mr. Brown:

On December 11, 1979 an engineer from our office accompanied the inspection team from SEA Consultants which was hired by the Corps of Engineers to inspect your dam, No. 191.02.

Although formal action by the Board on this matter will not be taken until receipt of the final report we thought you would be interested in the preliminary findings. A copy of our engineer's memo is enclosed for your information.

One item which you may wish to direct immediate attention to is the condition of the walkway over the right spillway. This structure is in very poor condition. Since your personal liability is at stake should someone be hurt, it would be in your best interest to either repair the catwalk, clearly post the area against trespassing, or block the area to access by the general public.

We will be in contact with you at some future date.

If you have any questions please contact us at your convenience.

Sincerely,

George McGee Sr

George M. McGee, Sr.,
Chairman

GMM:KS:paf
Enc.

M E M O

Date: December 12, 1979

To: Vernon A. Knowlton,
Chief Engineer

From: Ken Stern,
Water Resources Engineer KS

Subject: Corps Inspection of Noone Mills Dam, No. 191.02, Peterborough

On December 11, 1979 I accompanied the inspection team from SEA consultants. This is a stone and concrete dam with a sloping upstream timber face. The maximum height of the spillway is about 12 ft. The dam is located on the Contoocook River about a mile upstream of Peterborough. The pond is relatively small with portions greatly silted in. The dam is in POOR condition, the concrete is extremely deteriorated. The locations of major concrete deterioration are as follows:

1- <u>LEFT ABUTMENT</u>

a- Major deterioration at the end of the spillway crest.

b- Major deterioration and two leaks (10gpm ± each) at toe of abutment near spillway.

c- Crack and spalling on downstream side between spillway and penstock leaking less than 1gpm.

d- Severe spalling on downstream side at penstock.

e- Undercutting of upstream face just below waterline.

f- Crack in parapet wall directly over penstock.

2- <u>LONG LEFT PARAPET WALL</u>

a- Spalling along the waterline almost the entire length. Severe at some locations.

b- Numerous cracks, one of which is displaced about 1/2 inch.

c- Large pine tree growing adjacent to the wall.

d- Abundant bush growth along wall.

3- <u>SPILLWAY</u>

a- The downstream buttresses have lost their protective concrete covering.

b- Upstream wooden face appears deteriorated but could not be thoroughly inspected.

Date: December 12, 1979

To: Vernon A. Knowlton,
Chief Engineer

Subject: Noone Mills Dam Inspection (continued)

4- <u>RIGHT ABUTMENT</u>

a- At the toe next to the spillway there is a cavity large enough for a person to fit in (see photograph).

b- Severe spalling and cavitation at waste gate. Major leak (1cfs $\pm$) around right side of gate.

c- Minor seeps oozing through concrete.

5- <u>EMERGENCY SPILLWAY</u> (Flashboard Spillway)

This spillway was constructed in the area washed out by the 1938 flood. It appears to be in good condition.

a- Substantial debris has collected on the flashboards.

b- It appears that one board across the top may be missing.

c- The catwalk is missing many planks and the safety rail is wobbly.

BOTH THE PENSTOCK AND WASTE GATES APPEAR INOPERABLE DUE TO ROTTEN STEMS.

From records of the flood of 1938, the failure of the dam during low water would probably result in flow within the banks. During a major flood, the loss of the dam would increase the flood slightly for a short period.

I believe any action on this dam can wait until receipt of the Corps' report.

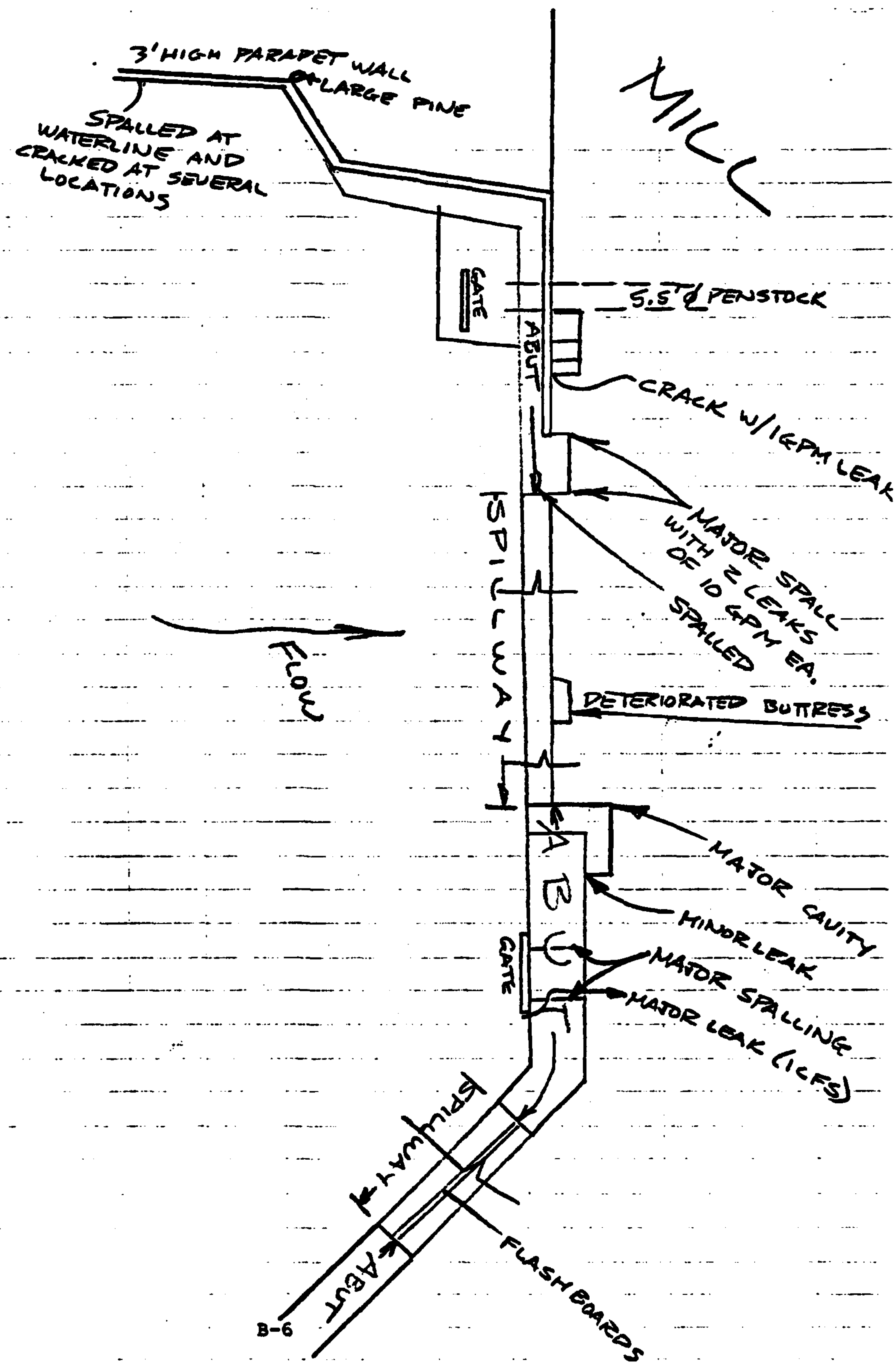
It was my impression that a senior engineer was not present. Those in attendance were from SEA: Bob Durfee (Party Chief), Bruce Pierstoff, Phil Upton, Phil Ricardi.

From GEI: Karl Dallenberg.

KJS:paf
Enc.

NOONE MILLS 191.02

STERN 12/12/79

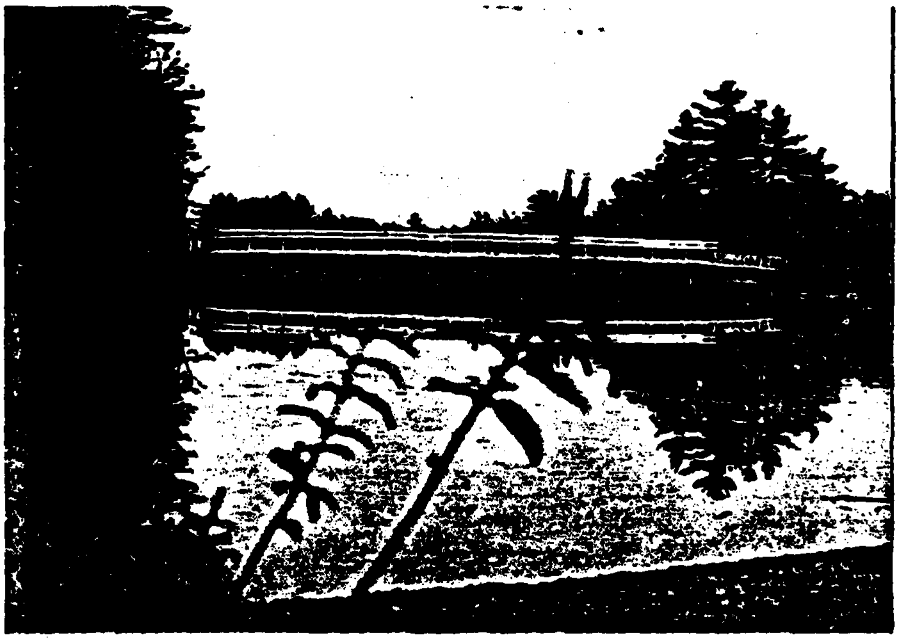


Army Corps of Engineers Dam Inventory Program

Dam # 191.02

Date August 28, 1974

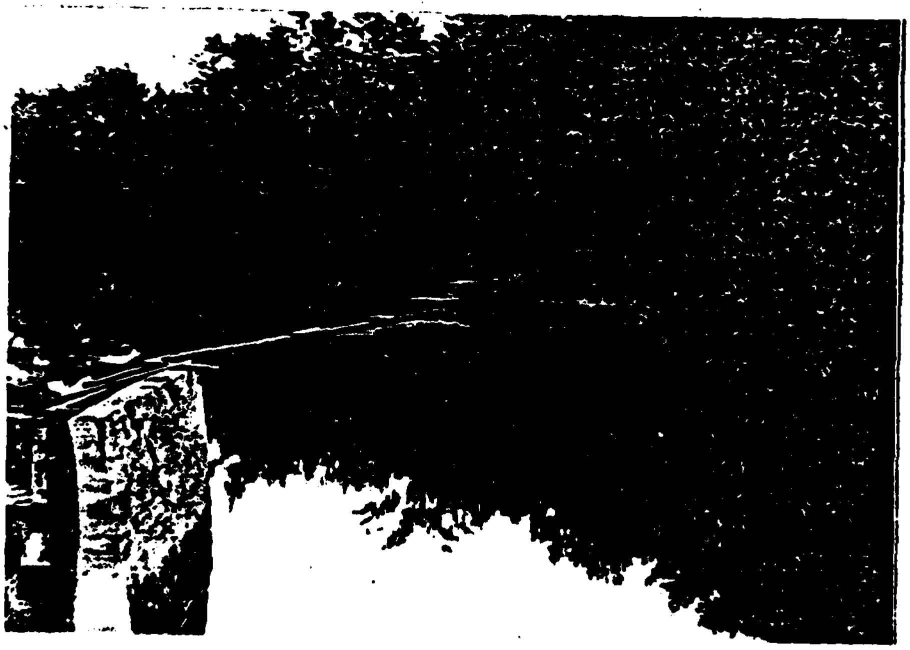
Corps # 427-191.02-32.09



Description: Impoundage

Dam # 191.02

Corps # 427-191.02-32.10



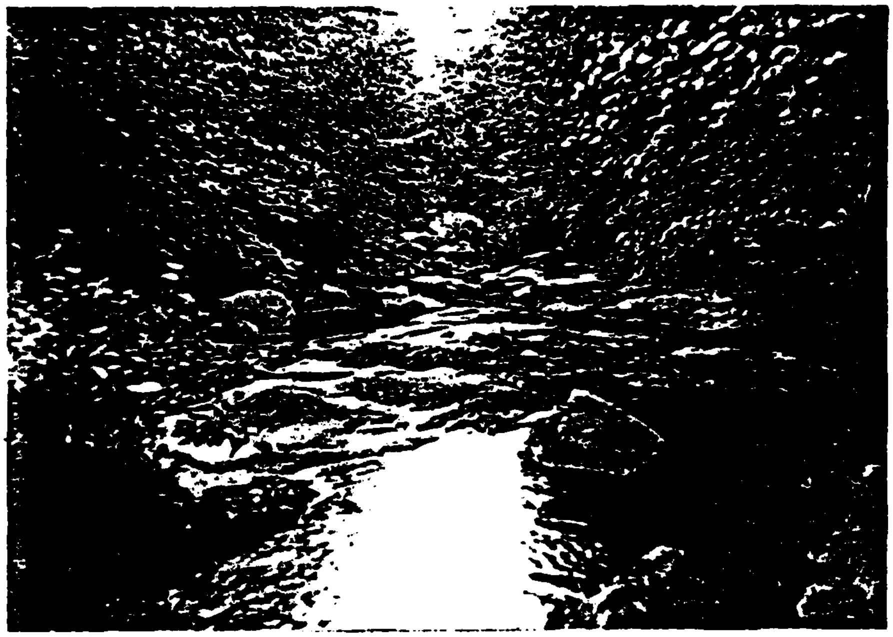
Description: Looking along upstream face of dam from right end

Army Corps of Engineers Dam Inventory Program

Dam # 191.02

Date August 28, 1974

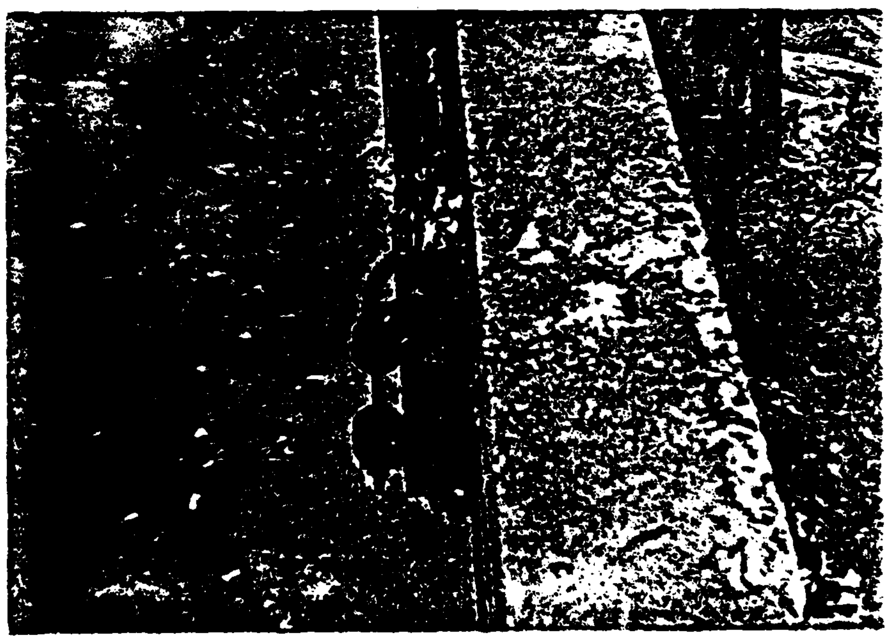
Corps # 427-191.02-32.07



Description: Looking downstream from emergency spillway

Dam # 191.02

Corps # 427-191.02-32.08



Description: Bags of mill waste used to slow leakage

Army Corps of Engineers Dam Inventory Program

Dam # 191.02

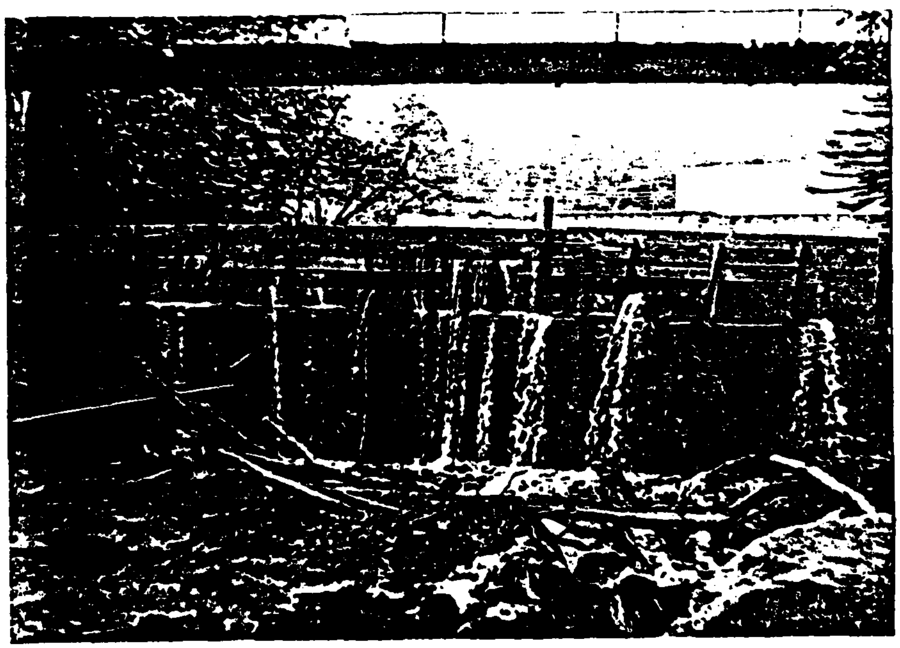
Date August 28, 1974

Corps # 427-191.02-32.05

Description: Typical spalled section of face

Dam # 191.02

Corps # 427-191.02-32.06



Description: Looking upstream at emergency spillway (location left end)

Army Corps of Engineers Dam Inventory Program

Dam # 191.02

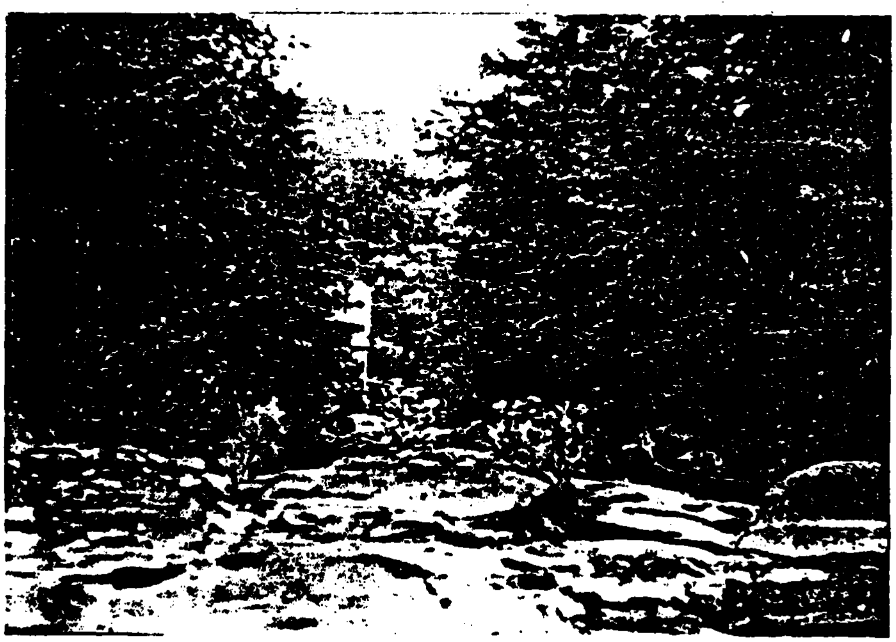
Date August 28, 1974

Corps # 427-191.02-32.03

Description: Looking upstream at right side of face

Dam # 191.02

Corps # 427-191.02-32.04



Description: Looking downstream from toe of dam

NEW HAMPSHIRE WATER CONTROL COMMISSION
DATA ON DAMS IN NEW HAMPSHIRE

LOCATION STATE NO. 191.02

Town Peterboro : County Hillsboro

Stream Contoocook R.

Basin-Primary Merrimack R. : Secondary Contoocook R.

Local Name N Noones Dam

Coordinates—Lat. 42° 50' + 9600 : Long. 72° 0' - 10300

GENERAL DATA

Drainage area: Controlled .... Sq. Mi.: Uncontrolled .... Sq. Mi.: Total 68.8 Sq. Mi.

Overall length of dam 236 ft.: Date of Construction 1831 mill started, 1923 cemented

Height: Stream bed to highest elev. 16.5 ft.: Max. Structure 14 ft.

Cost—Dam .... : Reservoir ....

DESCRIPTION Concrete wing wall left bank built after 1938 flood

Waste Gates

Type ....

Number 1 : Size 4 ft. high x 4 ft. wide

Elevation Invert .... : Total Area 16 sq. ft.

Hoist ....

Waste Gates Conduit

Number .... : Materials ....

Size .... ft.: Length .... ft.: Area .... sq. ft.

Embankment

Type concrete wall

Height—Max. .... ft.: Min. .... ft.

Top—Width .... : Elev. .... ft.

Slopes—Upstream .... on .... : Downstream .... on ....

Length—Right of Spillway 300 ± : Left of Spillway ....

Spillway

Materials of Construction concrete

Length—Total 164' ft.: Net .... ft.

Height of permanent section—Max. 14 ft.: Min. 10 ft.

Flashboards—Type Removable : Height .... ft.

Elevation—Permanent Crest .... : Top of Flashboard ....

Flood Capacity 5200 cfs.: 75.5 cfs/sq. mi.

Abutments

Materials: Mass concrete

Freeboard: Max. 6'-4 R.H. ft.: Min. 2.5' L.H. ft.

Headworks to Power Devel.—(See "Data on Power Development")

OWNER Jos. Noone Sons Co. (A.E. Goyette Prop.) Peterboro NH

REMARKS Left bank washed out Sept. 1938, wing wall at left bank built April 1939

Tabulation By C.F.C. Date April 28, 1939

B4821294

Route R. Co.

MEMORANDUM

Case No. C79-C

TO; Richard S. Holmgren, Chief Engineer

RE: Dam of Jos. Noone & Sons in Peterborough

As of your instructions of November 9, I visited the superintendent of the Noone Mills and Mr. Stevens, Superintendent of the Iafolla Construction Company and made the following changes:

The spill section on the south side of the dam was lowered to give a 4 foot freeboard between that section and the crest of the present dam. The abutment on the south side was raised and extended about 4 feet further into the bank giving a longer cutoff wall. This should adequately take care of any future flood flows. The section ~~will be~~ IS a very good section ~~if~~ AND IS reinforced.

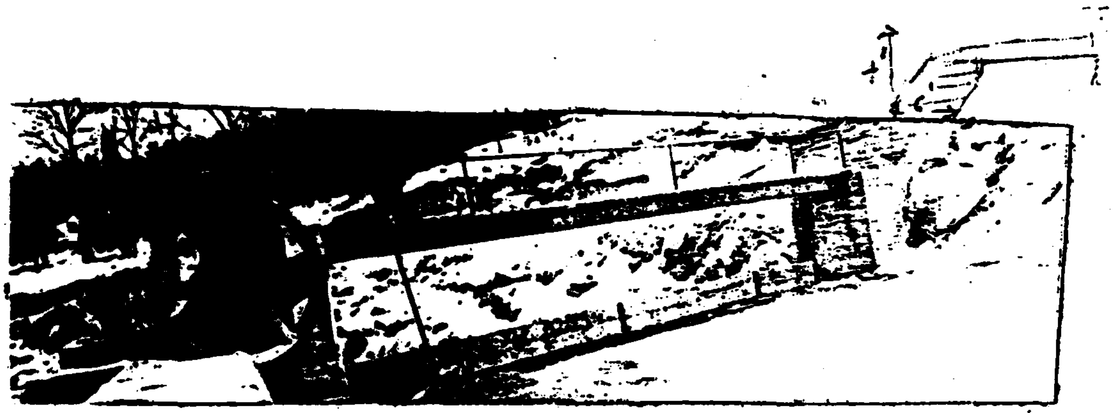
I again visited the site on Friday November 11 when they were starting to pour cement. The ledge was well cleaned of old shale and dowels driven in every three feet to a depth of 18 inches and well grouted into the ledge. I requested that the old section where adjacent to the new be well chipped and the ledge was well scrubbed before cement

was poured. A 1 - 2 - 4 - mix was used which is a standard 28 day 3000 pound test. mix The sand and aggregate was from a State tested bank and I believe that this should be a very good job. There was some trouble in handling water but Mr. Gayoutte let them PULL ~~peur~~ down the mill pond which helped. Sketches will be mailed later by Mr. Stevens.

Charles D. Colman

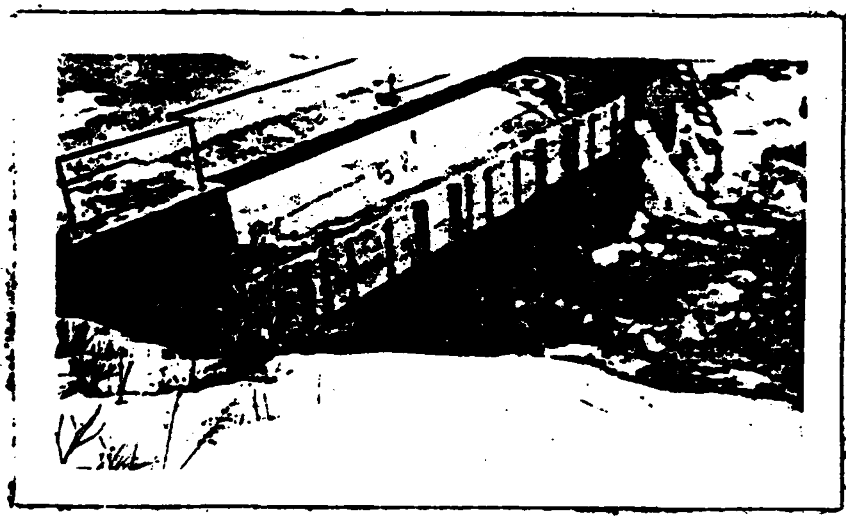
Charles D. Colman
Assistant Engineer

11/15/38



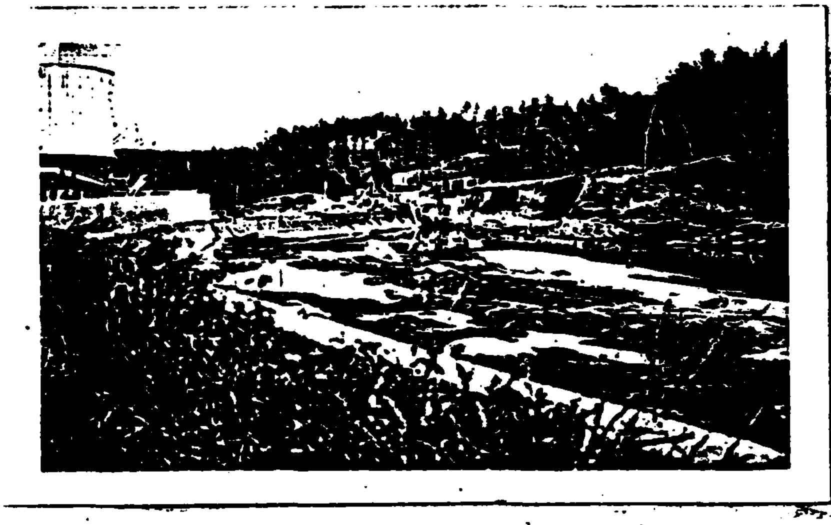
Joseph Noone & Sons Company Dam - Peterboro
Contoocook River   Additional section taken from South side.
Dam No. 191.02

4' BELOW CREST

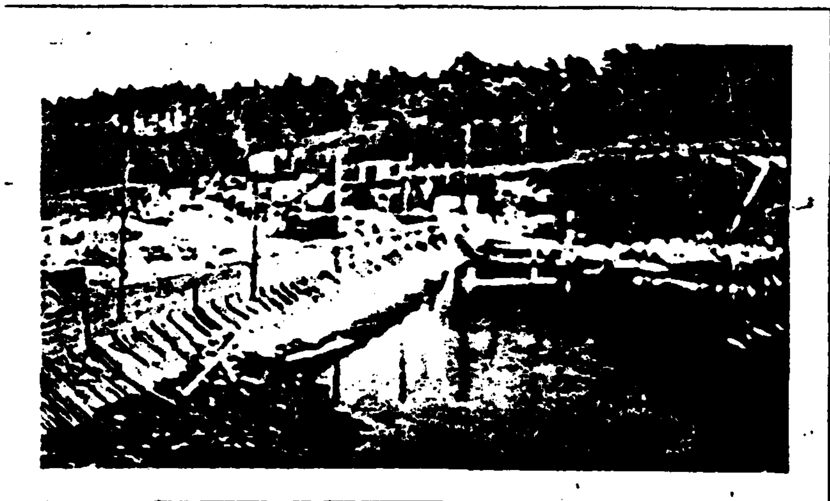


Joseph Noone & Sons Company Dam - Peterboro
Closeup view showing flashboards.

November 12, 1938



191.2--Peterboro
Noon Dam



191.2--Peterboro
Noon Dam

191.2--Peterboro
Noon Dam

Special delivery

WATER CONTROL COMMISSION

STATE OF NEW HAMPSHIRE

Concord, New Hampshire

October 7, 1938. Rec'd 10/11/38

Jacobson	
Holmgren	
Return to	
Filed	
File No.	

Jos. Noones & Sons Co.,
Peterboro, N. H.

RE: Noones Dam. W. C. C. No. 191.02

Gentlemen:

In order that we may determine the magnitude and extent of the flood of September 21-24 just passed, we are requesting the various dam owners in the State to supply us with the following information:

1. Was this dam injured? Ans. Yes
2. If so, to what extent? Ans. Bank on east end of dam washed away for 65 feet.
3. Did all flashboards go out? Ans. Yes
4. What was the maximum height of water over the permanent crest of spillway? Ans. 8 feet, 2 inches.
5. At what day and hour did the maximum flood height reach your dam? Ans. Sept. 21 between two and four p.m.
6. Any other interesting information regarding the flood or rain fall may be given on the back of this sheet, or attach sheets.

Will you please return this letter with as much information as you can give us as promptly as possible. A self-addressed envelope is attached hereto.

We thank you for your cooperation.

Over.

Very truly yours,

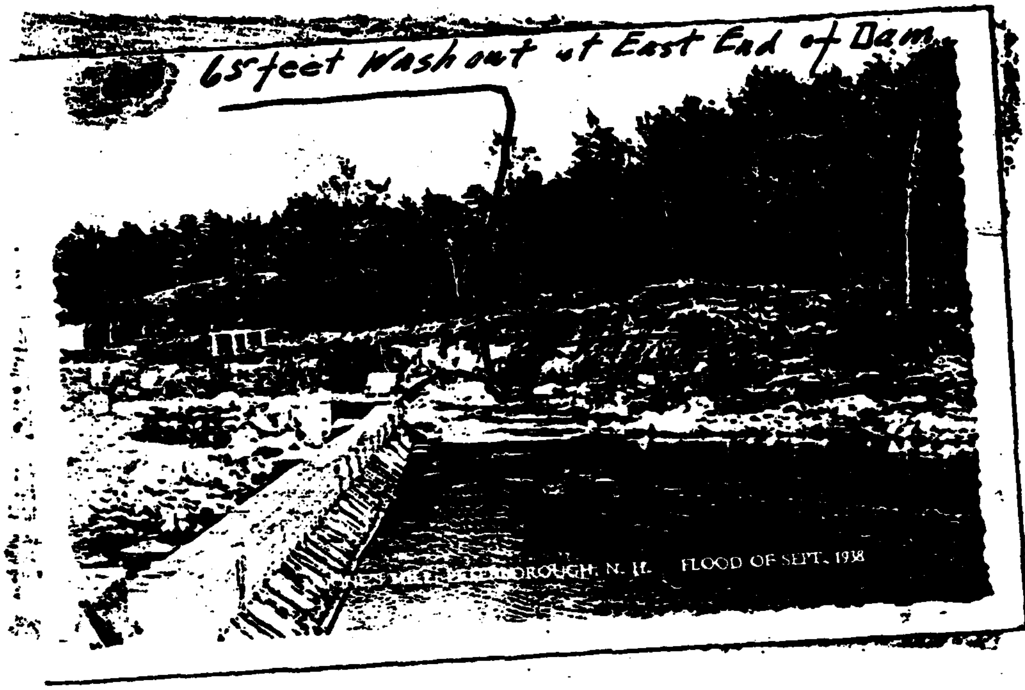
Richard S. Holmgren

Richard S. Holmgren
Chief Engineer

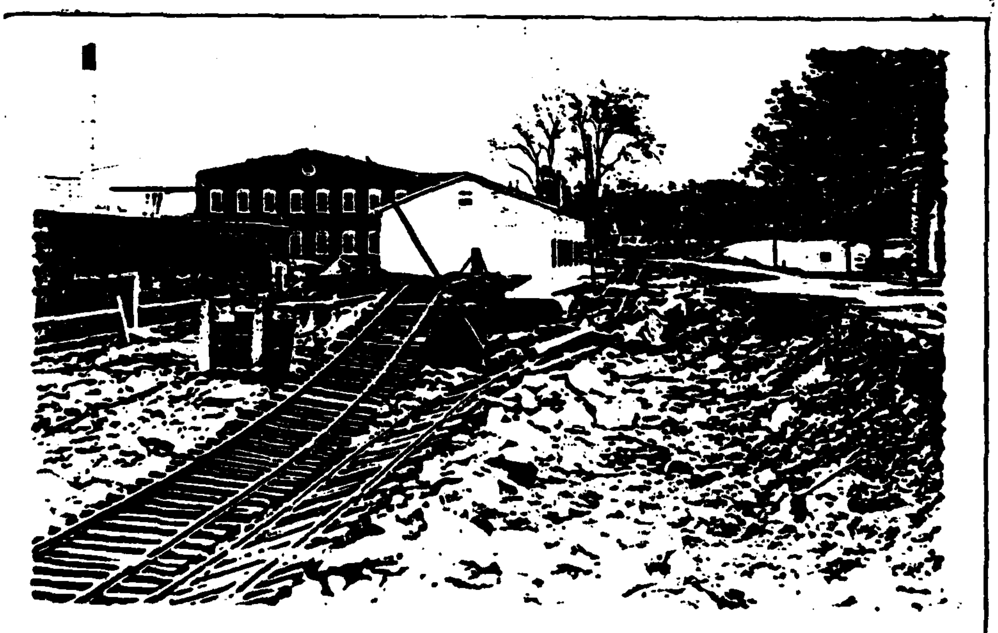
CDC:GMB
Enc.

B-16

(OVER)



Joseph Noone & Sons Company Dam - Peterborc
Dam No. 191.02



NEW HAMPSHIRE WATER RESOURCES BOARD

INVENTORY OF DAMS AND WATER POWER DEVELOPMENTS

DAM

BASIN MERRIMACK NO. *2 19.1.02 68.8-4/24/. ~~100 USGS~~
RIVER CONTOOCOOK MILES FROM MOUTH 160.2 D.A.SQ.MI ~~10.0 WAB~~
TOWN PETERBORO OWNER JOSEPH NOONE SONS CO., (A.E. GOYETTE PROP.)
LOCAL NAME OF DAM NOONES DAM
BUILT (1923 CEMENTED) DESCRIPTION CONCRETE FACE OLD STONE AND PLANK DAM
1831 MILL STARTED. CONCRETE WING WALL - LEFT BANK - BUILT 1936 AFTER FLOOD

POND AREA-ACRES DRAWDOWN FT. POND CAPACITY-ACRE FT.
HEIGHT-TOP TO BED OF STREAM-FT. 16.5 MAX. MIN.
OVERALL LENGTH OF DAM-FT. 236 MAX. FLOOD HEIGHT ABOVE CREST-FT.
PERMANENT CREST ELEV. U.S.G.S. LOCAL GAGE
TAILWATER ELEV. U.S.G.S. LOCAL GAGE
SPILLWAY LENGTHS-FT. 52' LOW - 112' MED - 9' HIGH FREEBOARD-FT. 2.5' RT. 6.3' LEFT
FLASHBOARDS-TYPE, HEIGHT ABOVE CREST

WASTE GATES-NO.	WIDTH	MAX. OPENING	DEPTH SILL BELOW CREST
1	4	4	(WASTE)
1			(WHEEL)

REMARKS CONDITION GOOD.

42°-50' + 9630 FT
~~CO-ORDINATES FROM A.E. 42° 50' + 3250 YDS.~~
~~72° 00' + 4000 YDS.~~
72° 00' - 10300 FT

POWER DEVELOPMENT

UNITS NO.	RATED HP	HEAD FEET	C.F.S. FULL GATE	KW	MAKE
		22	U.S.G.S. LIST		
1	150				24" RODNEY-HUNT TWIN

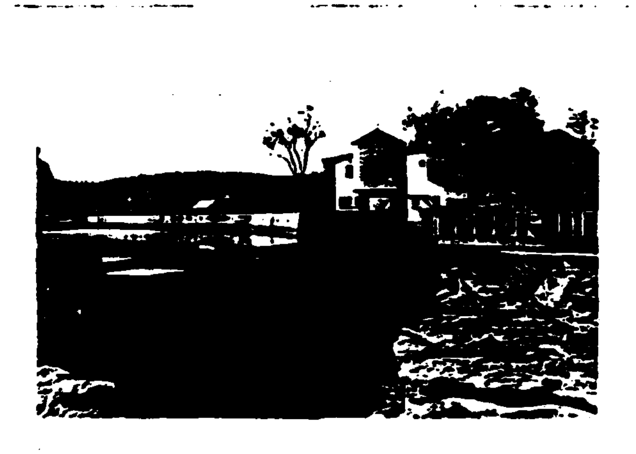
USE POWER FOR WOOLEN (FELT) MILL

REMARKS PRIMARY HP. 90% TIME 689 CFS SQ.MI. .37
ARMY INFORMATION FROM B. CUNNINGHAM C 155, 156, 157, 158, 159
6' STEEL PENSTOCK 100' ± SAW GEO. H. TRYON MASTER MECHANIC
INFORM. FROM ARNOLD RUNDLETT SUPT.

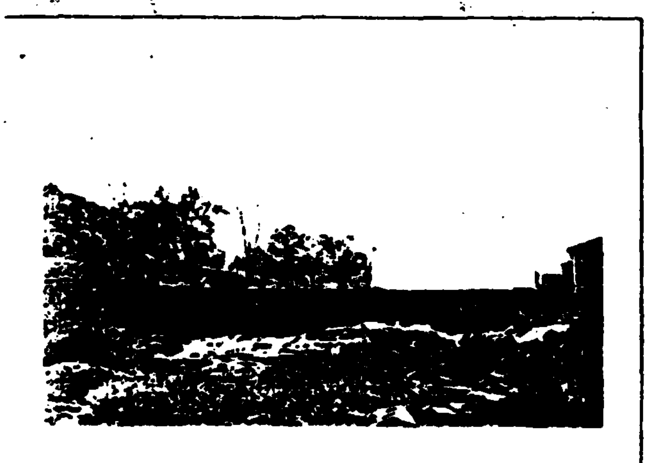
10-14-37
DATE 10-22/36 AE

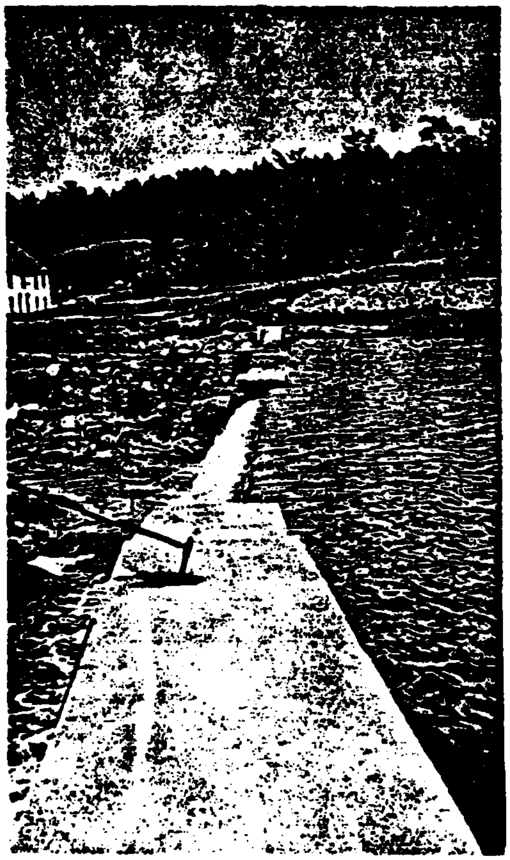
PLANS WITH MR. STEVENS

CONTOOCOOK RIVER IN PETERBORO
Joseph Noone & Sons
October 14,1937



Waste Gate

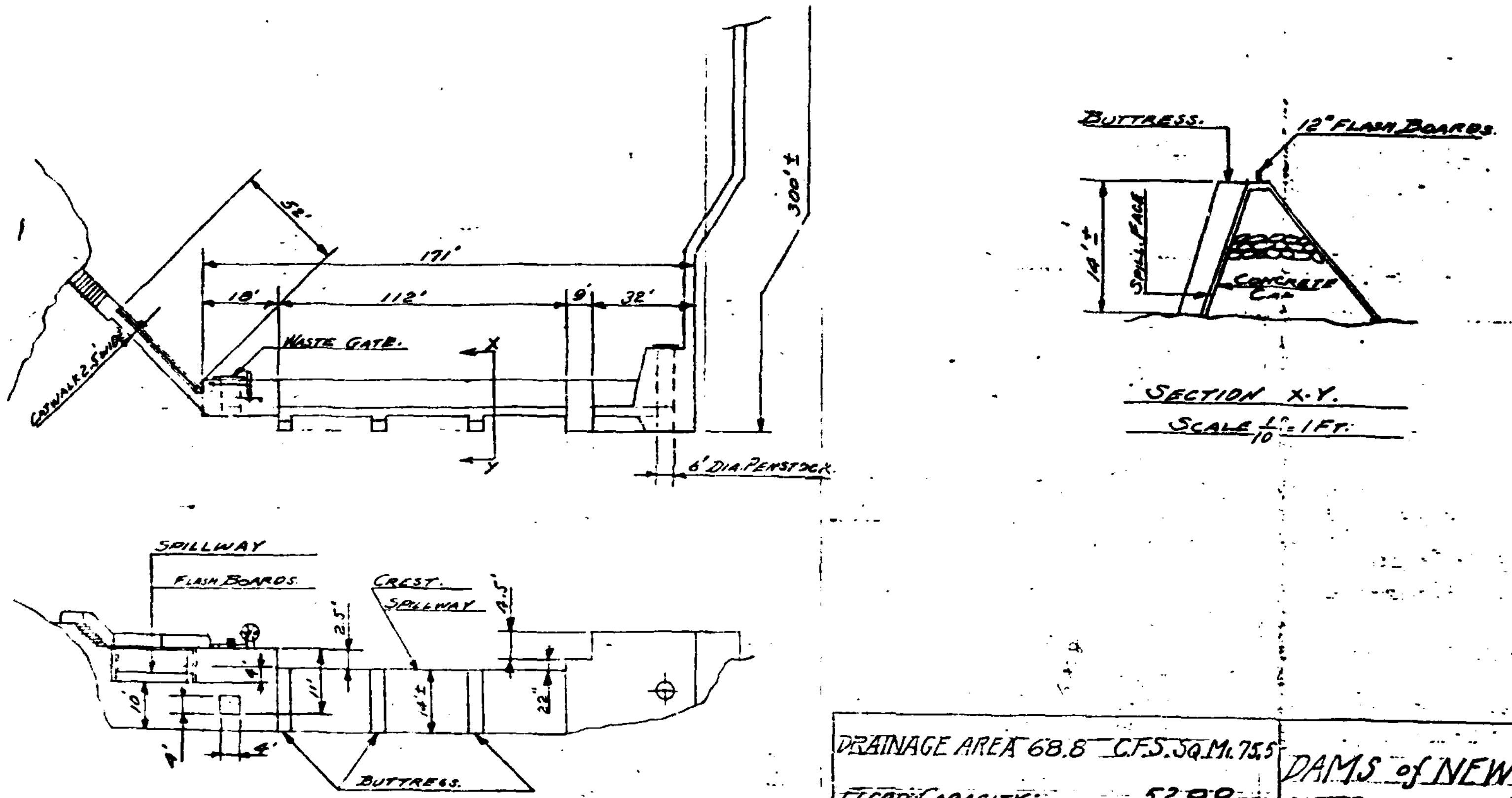




DWI-52 taken from the west wing wall on the gate hou[illegible]

PLANS AND DETAILS

DAM NO. 191.02



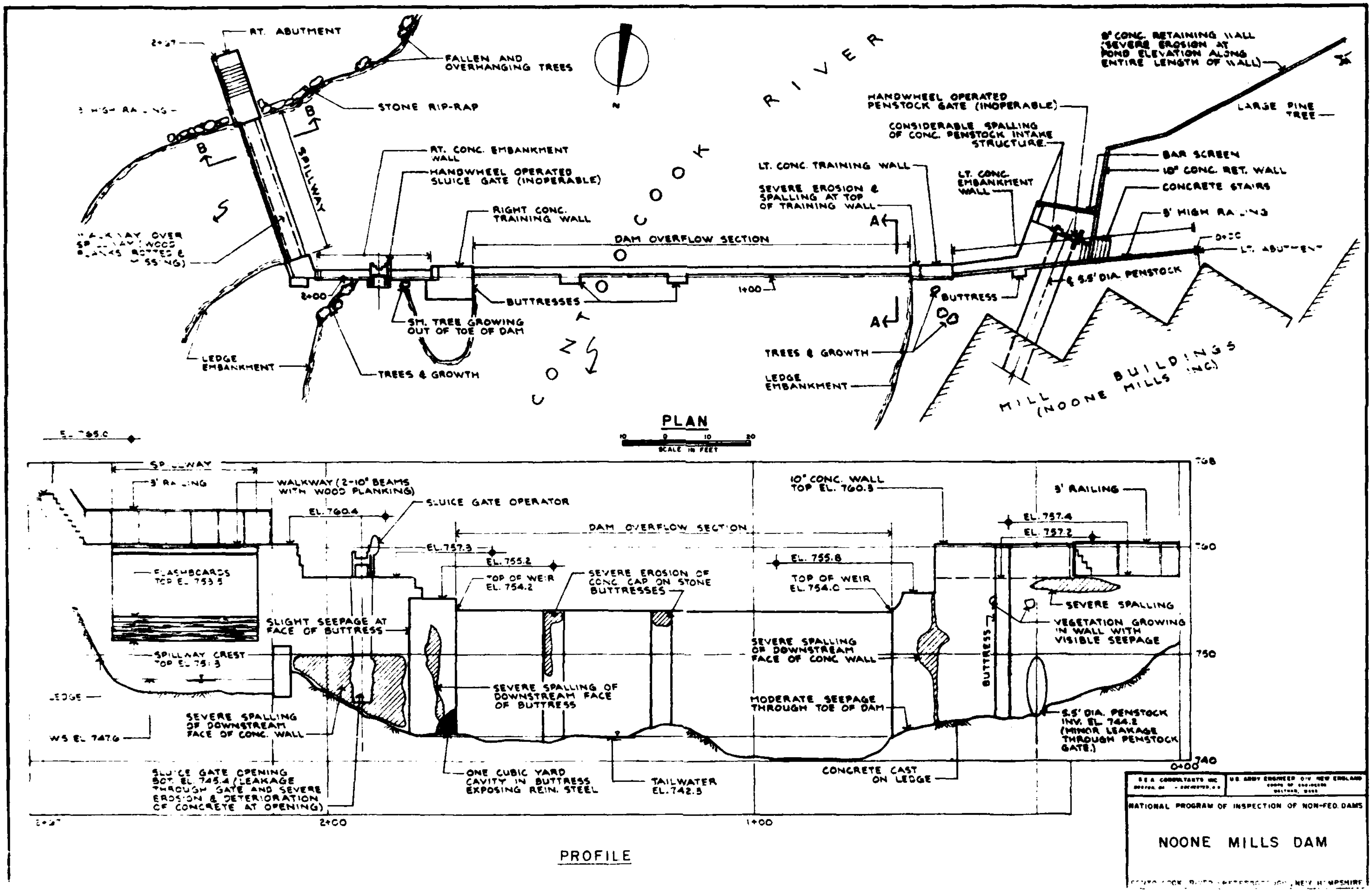
DRAINAGE AREA 68.8	C.F.S. SQ.MI. 75.5
FLOOD CAPACITY	5200
SPILLWAY	4930
GATE	270
TURN WHEEL	108
TOTAL	

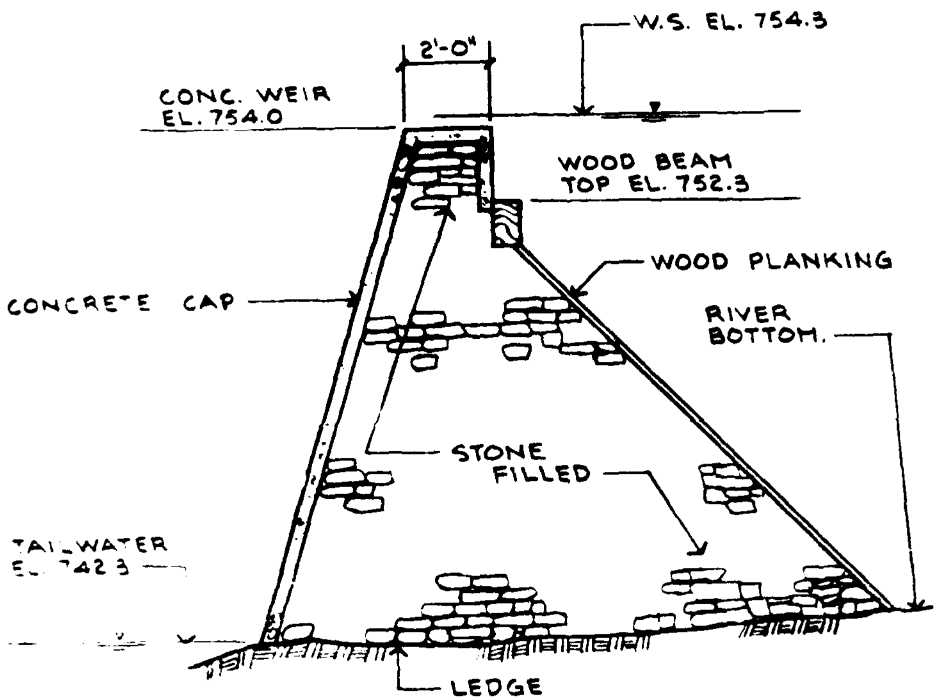
DAMS of NEW HAMPSHIRE
CONTOOCOOK RIVER
PETERBORO
PLAN & SECTION.
NEW HAMPSHIRE
WATER CONTROL COMMISSION
CONCORD, N.H.

DATE 4/24/39

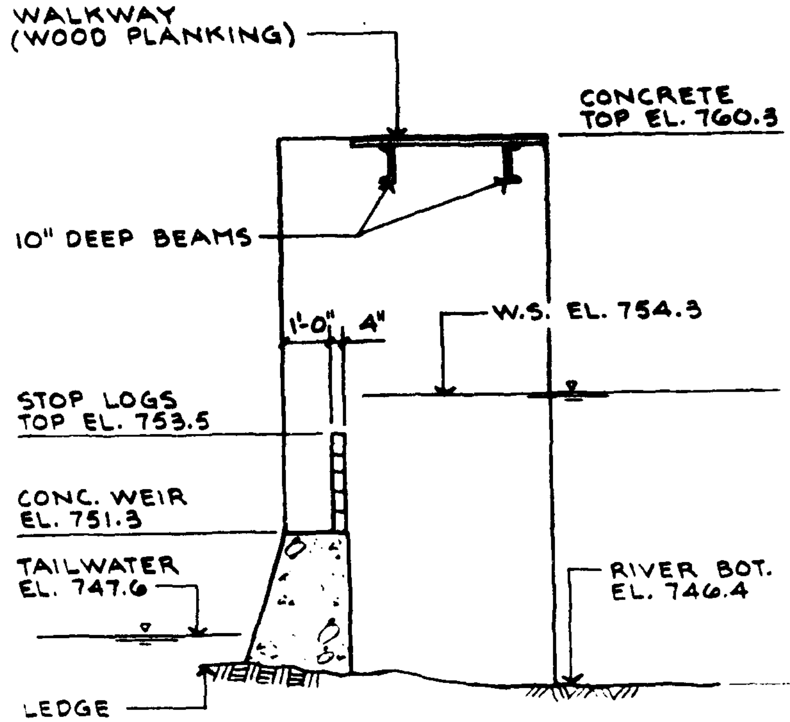
10 0 20 40 60 80

NO. 1
DRAWN BY C.F.O.


RT. ABUTMENT
FALLEN AND OVERHANGING TREES
STONE RIP-RAP
SPILLWAY
WALKWAY OVER SPILLWAY (WOOD PLANKS ROTTED & MISSING)
LEDGE EMBANKMENT
TREES & GROWTH
RT. CONC. EMBANKMENT WALL
HANDWHEEL OPERATED SLUICE GATE (INOPERABLE)
RIGHT CONC. TRAINING WALL
SM. TREE GROWING OUT OF TOE OF DAM
BUTTRESSES
DAM OVERFLOW SECTION
CONTOOCOOK RIVER
LT. CONC. TRAINING WALL
SEVERE EROSION & SPALLING AT TOP OF TRAINING WALL
TREES & GROWTH
LEDGE EMBANKMENT
HANDWHEEL OPERATED PENSTOCK GATE (INOPERABLE)
CONSIDERABLE SPALLING OF CONC. PENSTOCK INTAKE STRUCTURE
LT. CONC. EMBANKMENT WALL
BUTTRESS
8' CONC. RETAINING WALL (SEVERE EROSION AT POND ELEVATION ALONG ENTIRE LENGTH OF WALL)
LARGE PINE TREE
BAR SCREEN
10' CONC. RET. WALL
CONCRETE STAIRS
3' HIGH RAILING
LT. ABUTMENT
5.5' DIA. PENSTOCK
MILL BUILDINGS (NOONE MILLS INC.)
2+00
1+00
PLAN
SCALE IN FEET
SPILLWAY
3' RAILING
WALKWAY (2-10" BEAMS WITH WOOD PLANKING)
EL. 760.4
SLUICE GATE OPERATOR
EL. 757.3
EL. 755.2
TOP OF WEIR EL. 754.2
FLASHBOARDS TOP EL. 753.5
SLIGHT SEEPAGE AT FACE OF BUTTRESS
SPILLWAY CREST TOP EL. 751.3
LEDGE
WS EL. 747.6
SEVERE SPALLING OF DOWNSTREAM FACE OF CONC. WALL
SLUICE GATE OPENING BOT. EL. 745.4 (LEAKAGE THROUGH GATE AND SEVERE EROSION & DETERIORATION OF CONCRETE AT OPENING)
SEVERE EROSION OF CONC. CAP ON STONE BUTTRESSES
SEVERE SPALLING OF DOWNSTREAM FACE OF BUTTRESS
ONE CUBIC YARD CAVITY IN BUTTRESS EXPOSING REIN. STEEL
TAILWATER EL. 742.5
10' CONC. WALL TOP EL. 760.3
EL. 755.8
TOP OF WEIR EL. 754.0
SEVERE SPALLING OF DOWNSTREAM FACE OF CONC WALL
MODERATE SEEPAGE THROUGH TOE OF DAM
CONCRETE CAST ON LEDGE
3' RAILING
EL. 757.4
EL. 757.2
SEVERE SPALLING
VEGETATION GROWING IN WALL WITH VISIBLE SEEPAGE
BUTTRESS
5.5' DIA. PENSTOCK INV. EL. 744.2 (MINOR LEAKAGE THROUGH PENSTOCK GATE.)
760
750
740
0+00
2+00
1+00
PROFILE
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
NOONE MILLS DAM
U.S. ARMY ENGINEER DIV. NEW ENGLAND



SECTION A-A
(DAM OVERFLOW SECTION)
SCALE: 3/8"=1'



SECTION B-B
(SPILLWAY)
SCALE: 3/8"=1'

NOTES:

1) THE ELEVATIONS SHOWN ARE BASED ON AN ELEVATION OF 754.0 EXTRAPOLATED FROM USGS QUADRANGLE SHEET ASSUMED TO BE POOL ELEVATION AT PERMANENT SPILLWAY CREST.

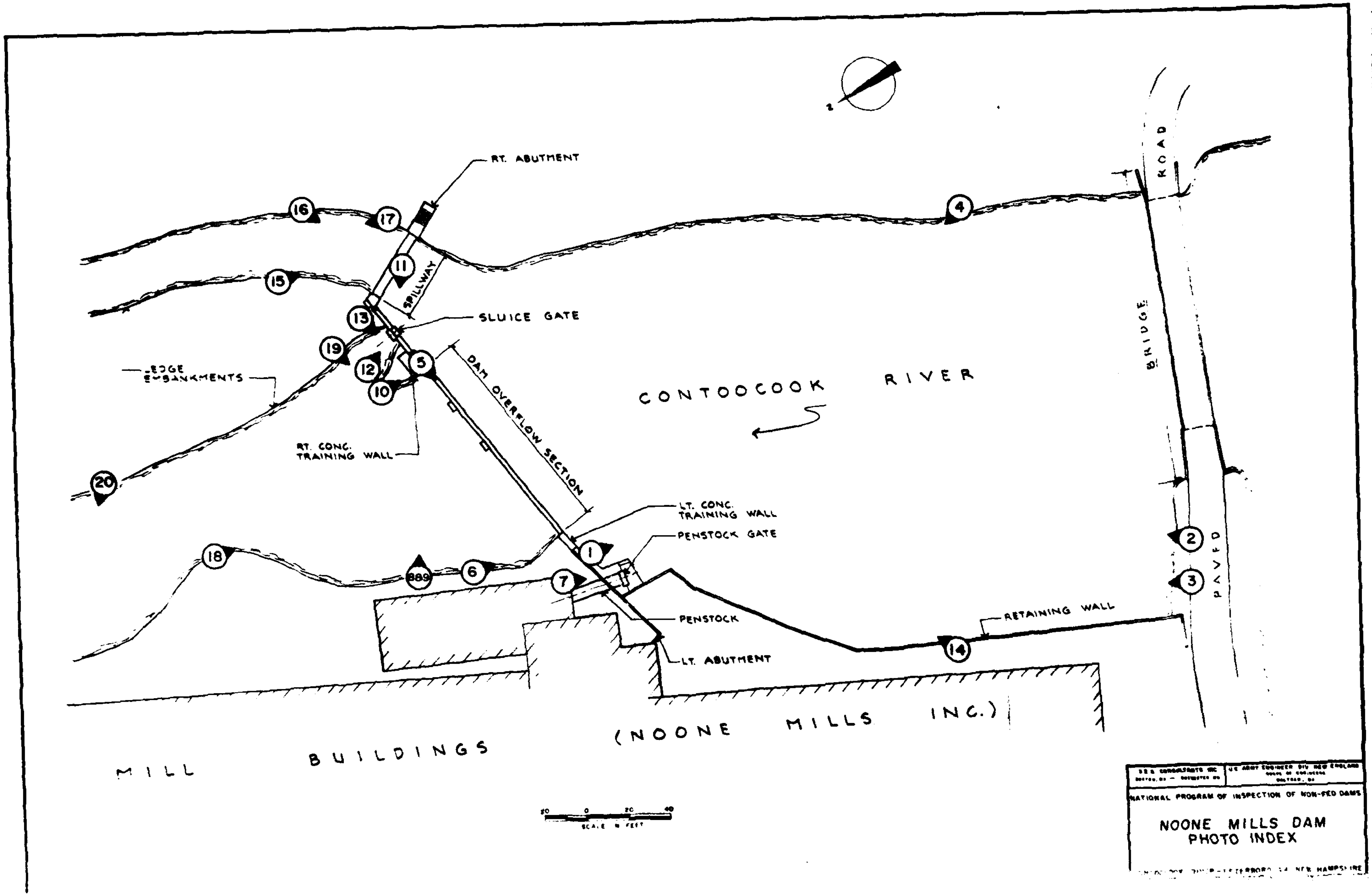
2) THE INFORMATION SHOWN ON THESE DRAWINGS ARE BASED ON EXISTING SKETCHES AND VISUAL OBSERVATIONS MADE DURING THE FIELD INSPECTION. DIMENSIONS OR MATERIALS INDICATED ON THESE DRAWINGS WHICH WERE BELOW GRADE OR WATER DURING THE TIME OF INSPECTION WERE NOT VERIFIED.

NATIONAL PROGRAM

NOONE MILLS DAM

APPENDIX C

SELECTED PHOTOGRAPHS



RT. ABUTMENT
SPILLWAY
SLUICE GATE
EDGE EMBANKMENTS
RT. CONC. TRAINING WALL
DAM OVERFLOW SECTION
CONTOOCOOK RIVER
LT. CONC. TRAINING WALL
PENSTOCK GATE
PENSTOCK
LT. ABUTMENT
RETAINING WALL
BRIDGE
ROAD
PAVED
MILL BUILDINGS (NOONE MILLS INC.)
SCALE IN FEET
20 0 20 40
1 2 3 4 5 6 7 8&9 10 11 12 13 14 15 16 17 18 19 20
NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS
NOONE MILLS DAM
PHOTO INDEX

Photo No. 1 - General view of upstream channel from dam.

Photo No. 2 - General view of dam from upstream channel.

Photo No. 3 - View of retaining wall on upstream face of left abutment.

Photo No. 4 - View of penstock intake structure from upstream channel.

Photo No. 5 - View of crest of overflow section and left abutment from right training wall.

Photo No. 6 - View of downstream face of left training wall of overflow section.

Photo No. 7 - Closeup view of vegetation and seepage on downstream face of penstock intake structure.

Photo No. 8 - View of right training wall of overflow section from downstream channel.

Photo No. 9 - Closeup view of base of right training wall of overflow section.

Photo No. 10 - Closeup view of large cavity in base of right training wall of overflow section.

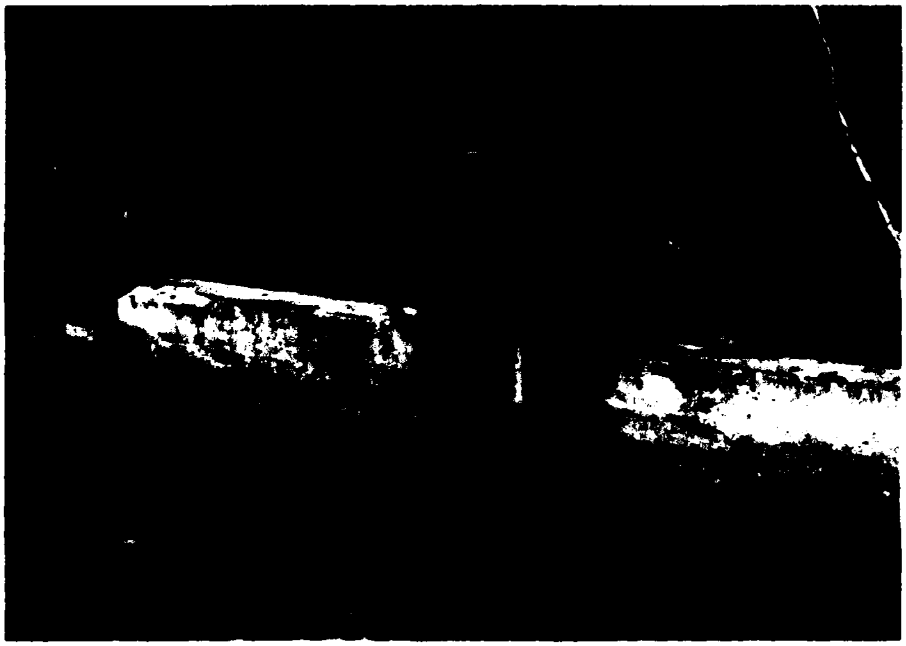


Photo No. 11 - Closeup view of sluicegate operator at right training wall of overflow section.

Photo No. 12 - Closeup view of sluicegate discharge opening at right training wall of overflow section.

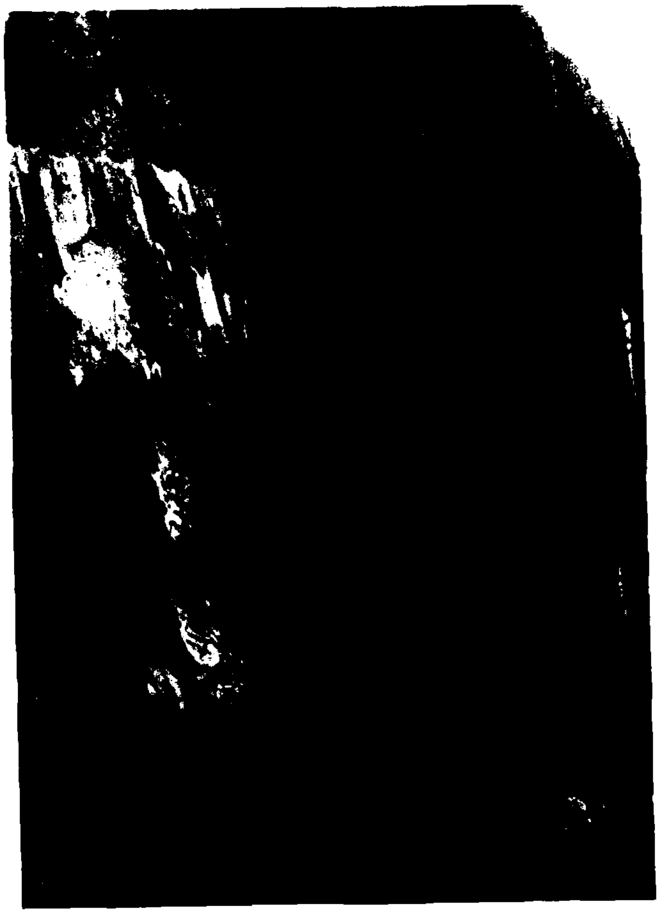


Photo No. 13 - Closeup view of vegetation and seepage on downstream face at right training wall of overflow section.

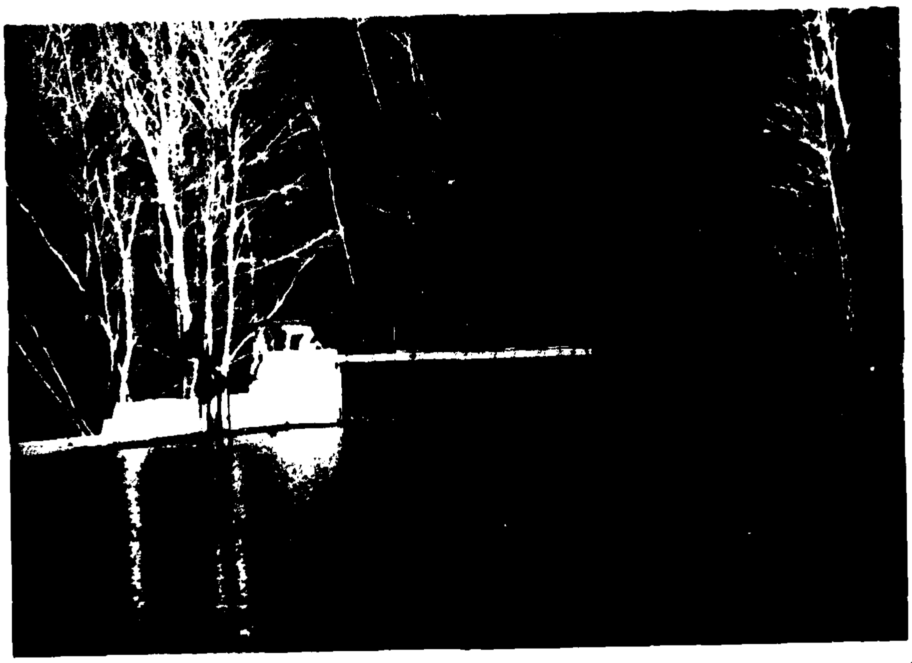


Photo No. 14 - General view of spillway from upstream channel.

Photo No. 15 - View of right abutment and spillway crest from downstream channel.

Photo No. 16 - View of downstream face of spillway.

Photo No. 17 - View of downstream channel below spillway.

Photo No. 18 - View of downstream face of overflow section.

Photo No. 19 - View of mill building immediately downstream of left abutment.

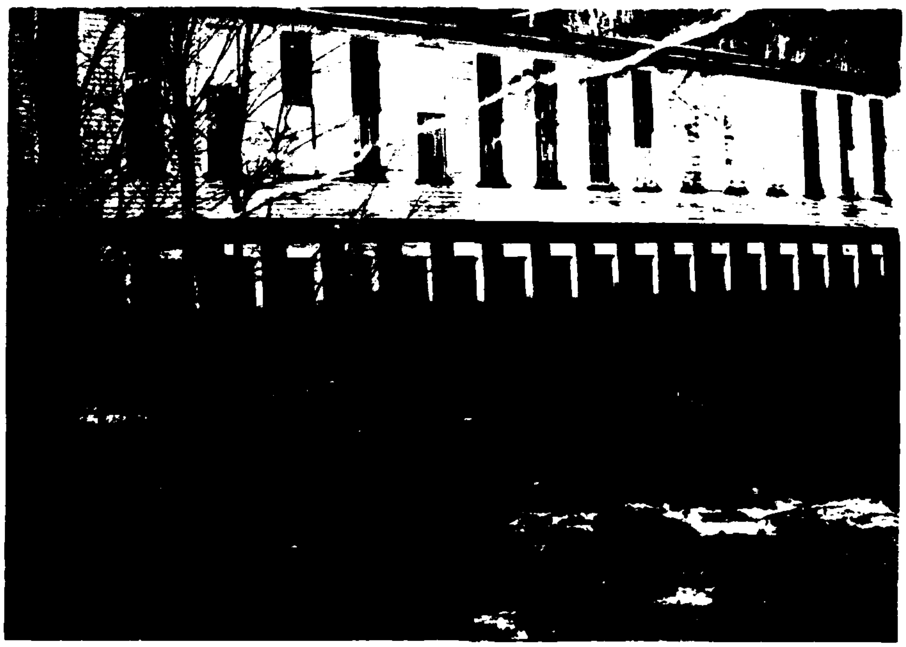


Photo No. 20 - View of mill building downstream of left abutment.

APPENDIX D

HYDROLOGIC AND HYDRAULIC COMPUTATIONS

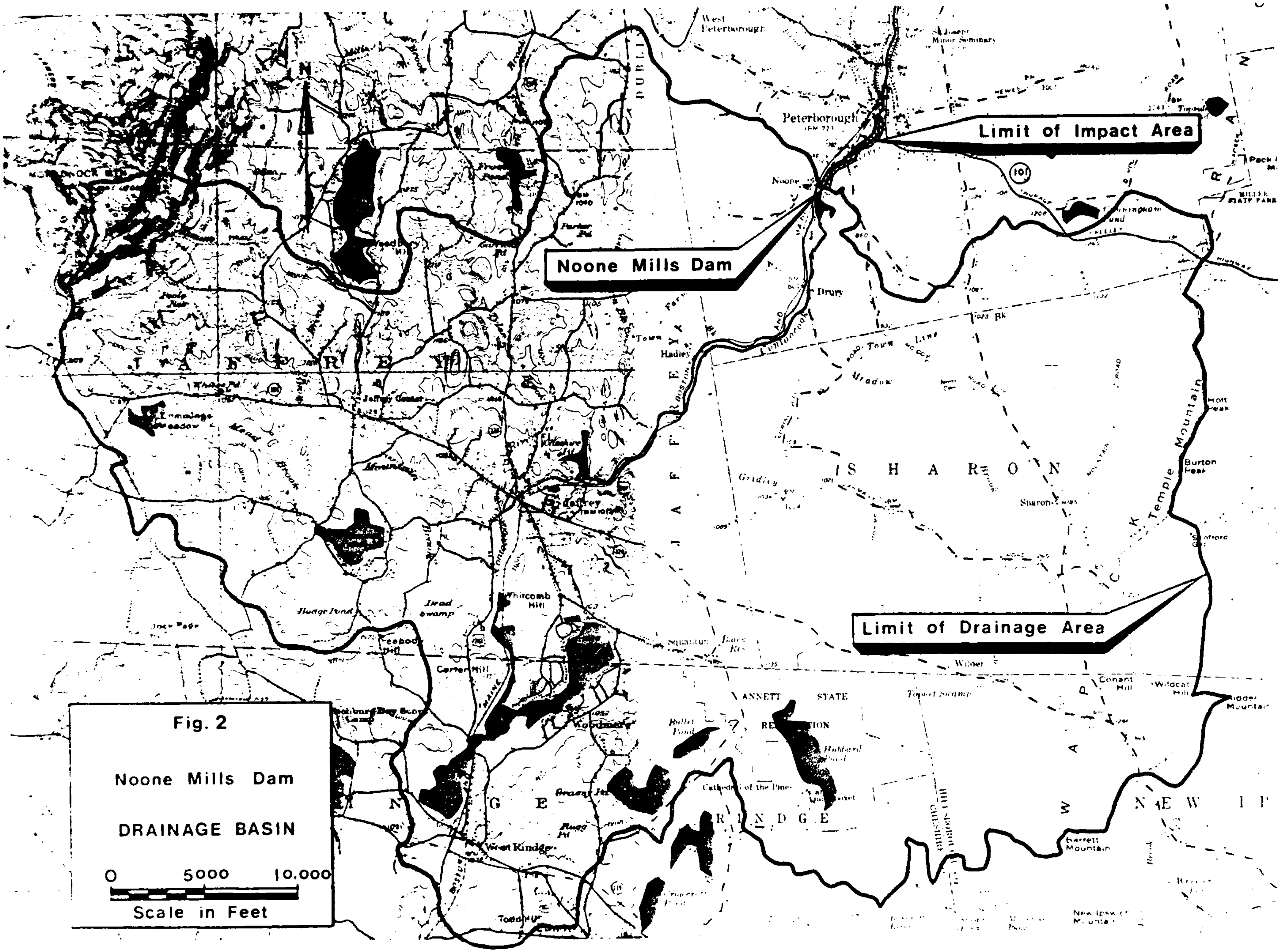


Fig. 2

Noone Mills Dam

DRAINAGE BASIN

0 5000 10,000

Scale in Feet

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ROCHESTER, N.H.

CLIENT Army Corps
JOB NO. 274-7901
PAGE 1 of 30
PROJECT Noone Mills Dam
COMPTD. BY BWP
DATE 2/26/90
DETAIL Hydrologic Calculations
CK'D. BY KMS
DATE 3/6/90

I. Basic Data

A. Drainage Area

1. 71.1 square miles - as defined on U.S.G.S. sheet and then planimetered

2. Drainage area has topography ranging from mountainous to rolling with numerous ponds in upper portion of drainage area.

B. Dam and Storage Information

1. Size Classification : SMALL based on storage (< 1000 acre-ft and ≥ 50 acre-ft)

   as indicated below - storage at crest of dam estimated to be 315 acre-ft

2. Hazard Potential : High hazard

   Damage to mill building and shopping center

3. Storage Information

Description of Information	Elevation * (feet)	Surface * Area (acres)	Storage * (acre-ft)
780 contour	780.0	137	
Test flood	767.0	72	635
Top of dam	760.3	38.6	315
760 contour	760.0	37	303

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PROJECT Noone Mills Dam
DETAIL Hydrologic Calcs

JOB NO. 274-7901
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CK'D. BY KMS

DATE 2/26/80
DATE 3/6/80

3. Storage Info - continued

Descriptive Information	Elevation	Surface Area	Storage
Crest of overflow section	754.0	19	135
Spillway - Top of flash boards	753.5	18.2	126
Spillway - permanent crest	751.3	14.6	89

Notes: (1) elevations: NGVD, - as interpreted from Peterborough Quadrangle map - elevation 754 taken to correspond to crest of overflow section

(2) pool shown on U.S.G.S. sheet taken to correspond with the elevation of the overflow section - ie 754.0

(3) surface area at crest of dam determined by interpolating between the surface areas defined by the 760 and 780 feet contours

C. Spillway Information

1 Principal spillway located at right abutment - Spillway has a length of 34 feet and permanent crest elevation of 751.3. Presently stoplogs have been installed on the spillway crest to an elevation of 753.5 A 4.0' wide by 4.5' high sluicegate is located in the dam face to the left of the spillway (between the spillway and overflow section). This gate is normally closed and is presently leaking -

a. for the surcharge storage calculations it was assumed that the sluicegate is closed and that the flashboards will remain in place

2. Discharge over the spillway may be determined with the sharp-crested weir equation

$$Q = C L H^{3/2}$$

where: Q = discharge, cfs
C = discharge coefficient
L = length of weir, feet
H = head over weir, feet

II. Estimate Effect of Surcharge Storage on Maximum Probable Discharge

A. Develop stage - discharge curve for outflow from the dam complex

1. define sources of outflow -

a discharge over spillway - above elevation 753.5 as defined above

b. discharge over overflow section of dam - above elevation - 754.0' - use broad crested weir equation

c. discharge over various training walls, platforms, etc - starting above elevation 755.2' - use broadcrested weir equation

d discharge over abutments - starting above elevation 760.0' - use broadcrested weir equation

2. Discharge over spillway

Elevation, feet	C	L, feet	H, feet	Q cfs
753.5	—	—	0	0
754	3.30	34	0.5	40
755	3.35		1.5	209
756	3.41		2.5	458
758	3.52		4.5	1,140
760	3.63		6.5	2,050
762	3.74		8.5	3,150
764	3.86		10.5	4,470
766	3.97		12.5	5,970
768	4.08		14.5	7,660
770	4.19		16.5	9,550
772	4.30		18.5	11,600
774	4.41		20.5	13,900
776	4.53	34	22.5	16,400

3. Discharge over overflow section

Elevation, feet	C	L, feet	avg. H, feet	Q cfs
754.0	—		—	0
755	2.9		0.9	252
756	2.9	102	1.9	775
758	3.3		3.9	2590
760			5.9	4820
762			7.9	7470
764			9.9	10,500
766			11.9	13,800
768			13.9	17,400
770			15.9	21,300
772			17.9	25,500
774			19.9	29,900
776	3.3	102	21.9	34,500

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DETAIL Hydrologic Calcs　CK'D. BY KMS　DATE 3/6/80

4. Discharge over various training walls, operator platforms, etc

a. right overflow section training wall

Elevation, feet	C	L feet	H, feet	Q cfs
755.2	—	—	—	0
756	2.7	8	0.8	15
758	2.6		2.8	97
760			4.8	219
762			6.8	369
764			8.8	543
766			10.8	738
768			12.8	952
770			14.8	1190
772			16.8	1430
774			18.8	1690
776	2.6	8	20.8	1970

b. left overflow section training wall

Elevation feet	C	L. feet	H feet	Q cfs
755.8	—	—	—	0
756	24	10	0.2	2
758	27		2.2	88
760	28		4.2	240
762	3.3		6.2	503
764			8.2	773
766			10.2	1,070
768			12.2	1,400
770			14.2	1,700
772			16.2	2,150
774			18.2	2,500
776	3.3	10	20.2	2,990

c. sluice gate operator platform between spillway and overflow section

Elevation feet	C	L feet	H feet	Q cfs
757.3	—	—	0	0
758	3.1	29	0.7	53
760	3.3	↑	2.7	425
762	↑		4.7	975
764			6.7	1660
766			8.7	2460
768			10.7	3350
770			12.7	4330
772			14.7	5,390
774	↓	↓	16.7	6,530
776	3.3	29	18.7	7,740

d. left and right spillway training walls

Elevation feet	C	Total L (both walls) feet	H feet	Q cfs
760.4	—	—	0	0
762	2.7	16	1.6	87
764	2.7	↑	3.6	295
766	2.8		5.6	594
768	↑		7.6	939
770			9.6	1,330
772			11.6	1,770
774	↓	↓	13.6	2,250
776	2.8	16	15.6	2,760

e. wall adjacent to left abutment

Elevation, feet	C	L, feet	H feet	Q cfs
760.3	—	—	—	0
762	3.3	33	1.7	241
764	↑	↑	3.7	775
766			5.7	1430
768			7.7	2320
770			9.7	3230
772			11.7	4350
774	↓	↓	13.7	5520
776	3.3	33	15.7	6770

f. stairs adjacent to right abutment

Elevation feet	C	L feet	Avg. H feet	Q cfs
760.3	-	—	—	0
762	2.6	2.6	0.85	5
764	↑	6.0	1.85	39
766		7.0	3.35	112
768		↑	5.35	225
770			7.35	363
772			9.35	520
774	↓	↓	11.35	696
776	2.6	7.0	13.35	888

g. Summary of and total of discharges over various training walls, operator platforms, etc

3-significant digits

Elevation feet	Q_{4a}	Q_{4b}	Q_{4c}	Q_{4d}	Q_{4e}	Q_{4f}	Q SUB-TOTAL
755.2	0	0	0	0	0	0	0
756	15	2	0	0	0	0	17
758	97	88	53	0	0	0	238
760	219	240	425	0	0	0	884
762	369	508	975	87	241	5	2190
764	543	773	1,660	295	775	39	4090
766	733	1,070	2,460	594	1,480	112	6,450
768	952	1,400	3,350	939	2,320	225	9,190
770	1180	1,760	4,330	1,330	3280	363	12,200
772	1,430	2,150	5,390	1,770	4,350	520	15,600
774	1,690	2,560	6,530	2,250	5,520	696	19,200
776	1,970	2,990	7,740	2,760	6,770	893	23,100

5. Discharge over left abutment - discharge controlled by roadway upstream from dam - break into two segments (1) flow over relatively flat portion of roadway (2) the triangular "weir" section created by flow over inclined slope to west of roadway

a. discharge over flat section of roadway

Elevation feet	C	Total L feet	H feet	Q cfs
760.0	—	—	0	0
762	2.6	95	2	699
764	↓	↓	4	1980
766	↓	↓	6	3,630
768	↓	↓	8	5,590
770	↓	↓	10	7,810
772	↓	↓	12	10,300
774	↓	↓	14	12,900
776	↓	↓	16	15,300

b. discharge over inclined slope

Elevation feet	C	L feet	Avg H feet	Q cfs
760.0	—	—	—	0
762	2.6	24	1.0	62
764	↓	48	2.0	353
766		70	3.0	946
768		94	4.0	1950
770		118	5.0	3430
772		142	6.0	5430
774		164	7.0	7700
776		190	8.0	11,200

6 Discharge over right abutment

Elevation feet	C	L feet	Avg H feet	Q cfs
765.0	—	—	—	0
766	2.6	10	0.5	9
768	↓	28	1.5	132
770		47	2.5	480
772		66	3.5	1120
774		85	4.5	2100
776		104	5.5	4470

7. Total discharge from project site

3-significant digits

Elevation feet	Q spillway	Q overflow section	Q SUB-TOTAL	Q roadway left abut	Q slope left abut.	Q right abut.	Q TOTAL
753.5	0	0	0	0	0	0	0
754	40	0	0	0	0	0	40
755	209	252					461
756	458	775	17	0	0	0	1,250
758	1,140	2,590	233	0	0	0	3,970
760	2050	4,820	884	0	0	0	7,750
762	3,150	7,470	2,190	699	62	0	13 600
764	4,470	10,500	4,090	1,980	353	0	21,400
766	5,970	13,800	6,450	3,620	946	9	30,800
768	7,660	17,400	9,190	5,510	1,960	132	41,900
770	9,550	21,300	12,200	7,910	3,430	480	54 800
772	11,600	25,500	15,600	10,300	5,430	1120	69,600
774	13,900	29,900	19,200	12,900	7,900	2100	85,900
776	16,400	34,500	23,100	15,800	11,200	4470	105,000

Discharge vs Elevation summarized graphically in Figure 1

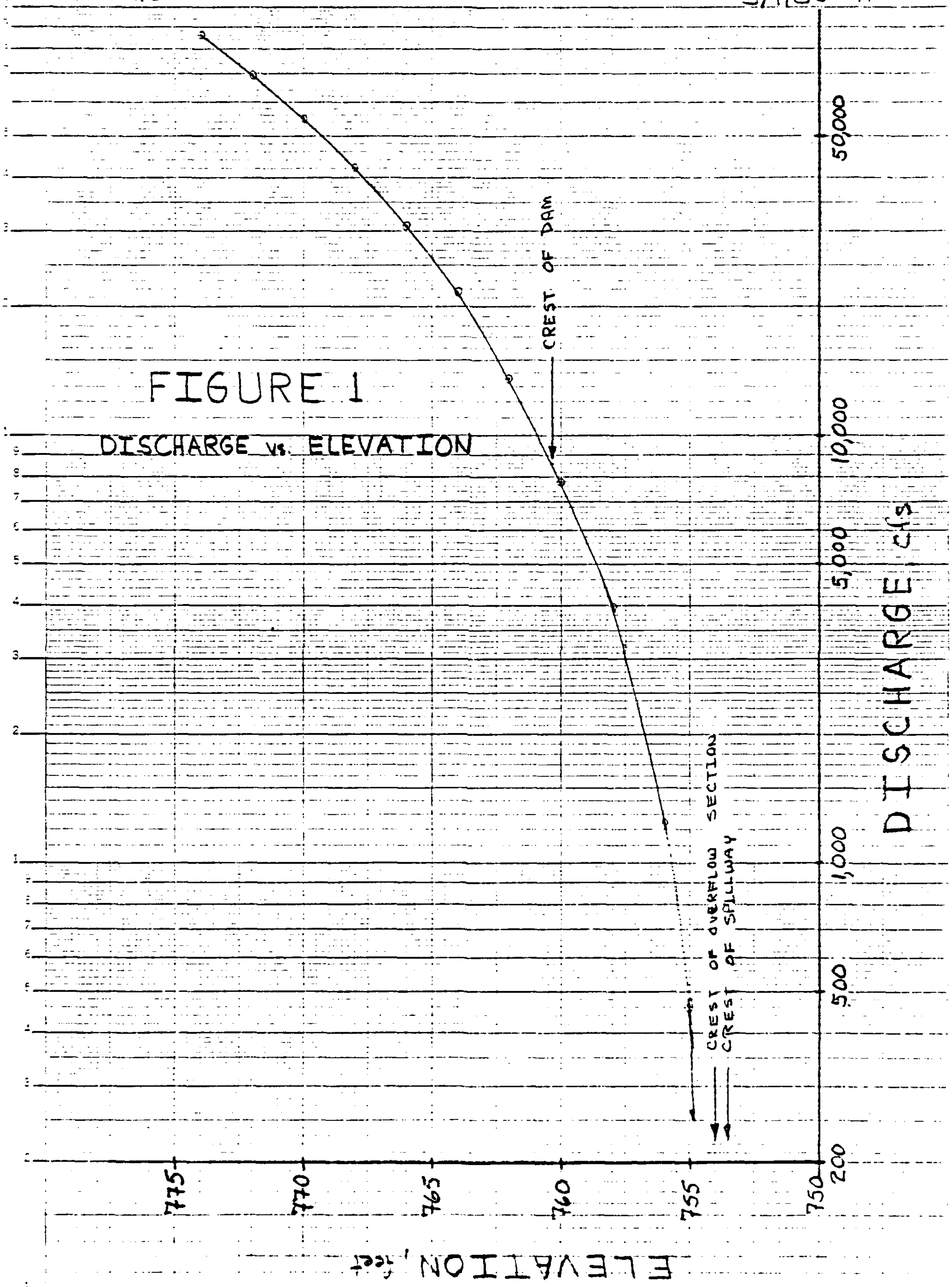

FIGURE 1

DISCHARGE vs. ELEVATION

B. Effect of surcharge storage on max. prob. discharge

1. Pertinent Data

   a. Drainage area = 71 square miles
   b. Characteristics of basin - ranges from mountainous to rolling with numerous ponding areas in upper portions of basin
   c. Test flood = use 1/2 PMF (small size and high hazard)
   d. Follow Army Corps' procedure

2. STEP 1: Determine Peak Inflow Q_{P1} from Guide Curve - will use rolling curve - assume that mountainous portions balanced by ponding areas thus reducing entire basin to the rolling curve

   a. the maximum probable discharge was estimated to be 1,075 cfs / sq mile

$$\therefore \text{PMF} = (1{,}075 \text{ cfs/sq. mile})(71.1 \text{ sq mile})$$

$$= 76{,}325 \text{ cfs}$$

$$1/2 \text{ PMF} \approx 38{,}200 \text{ cfs}$$

3. STEP 2: Determine surcharge height to pass Q_{P1}, $STOR_1$, and Q_{P2}

   a. from Figure 1 determine surcharge height to pass Q_{P1} = 38,200 cfs (assume initial water surface at top of stoplogs)

surcharge elevation = 767.5'
Top of flashboards = 753.5'
surcharge height = 14.0 feet

   b. determine volume of surcharge $STOR_1$ in inches of runoff

first determine volume of storage in acre-ft in following manner:
(1) determine surface area of pond corresponding to surcharge elevation from Figure 2 ≈ 74 acres
2 determine average surface area for surcharge height
(a) will be done in two steps since surface area vs elevation line does not have a constant slope
① between elevations 753.5' and 760.0'
② above elevation 760.0'

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3/4/80

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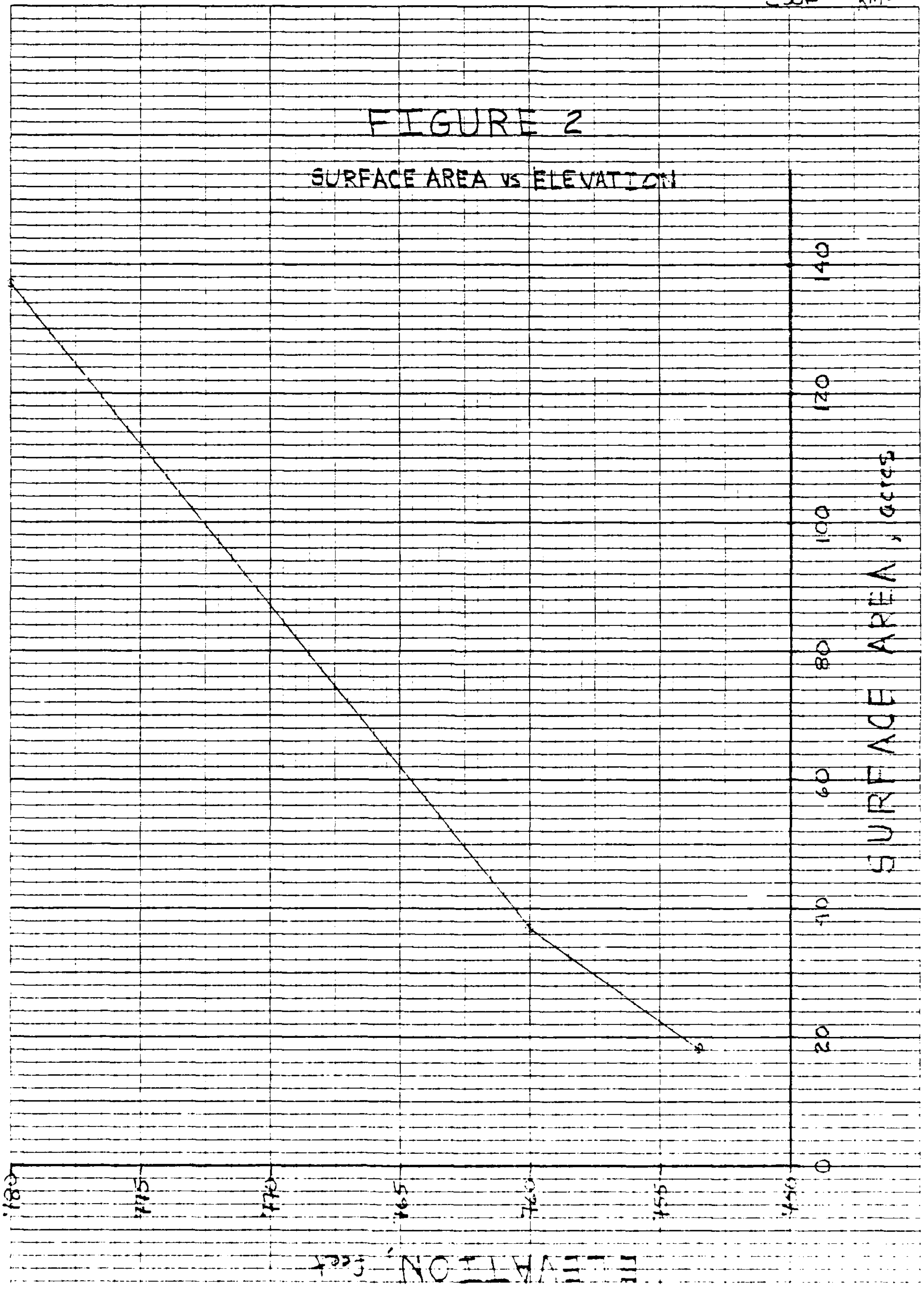

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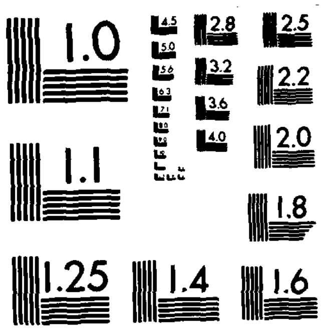

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(3) multiply each portion of the surcharge height by the corresponding average surface area to determine the volume of storage in acre-ft for input into the following equation

$$STOR_1 = \frac{\text{Volume of storage (as acre-inches)}}{\text{drainage area}}$$

$$STOR_1 = \frac{\left[(6.5ft)\left(\frac{18.2 \text{ acres} + 37 \text{ acres}}{2}\right) + (7.5ft)\left(\frac{37 \text{ acres} + 74 \text{ acres}}{2}\right)\right](12''/ft)}{(71 \text{ sq. mi})(640 \text{ acres/sq. mi})}$$

$$STOR_1 = 0.158 \text{ inches}$$

c. determine Q_{P2}

$$Q_{P2} = Q_{P1}\left(1 - \frac{STOR_1}{9.5}\right)$$ use 9.5" since considering 1/2 PMF

$$Q_{P2} = (38{,}200 \text{ cfs})\left(1 - \frac{0.158}{9.5}\right)$$

$$Q_{P2} = 37{,}600 \text{ cfs}$$

4. STEP 3: Determine surcharge height and $STOR_2$ to pass Q_{P2} and then Q_{P3}

a. From Figure 1 determine surcharge height to pass Q_{P2} = 37,600 cfs

Surcharge elevation = 767.0'
Top of flashboards = 753.5'
surcharge height = 13.5 feet

surface area at 767.0 = 72 acres

b. determine $STOR_2$

$$STOR_2 = \frac{\left[(6.5\,ft)\left(\frac{18.2\,ac + 37\,ac}{2}\right) + (7.0\,ft)\left(\frac{37\,ac + 72\,ac}{2}\right)\right](12''/ft)}{(71\,sq.mi)(640\,acres/sq.mi)}$$

$$= 0.148 \text{ inches}$$

c. Average $STOR_1$ and $STOR_2$

$$STOR_{AVG} = \frac{STOR_1 + STOR_2}{2}$$

$$STOR_{AVG} = \frac{0.158\text{ in.} + 0.148\text{ in.}}{2}$$

$$STOR_{AVG} = 0.153 \text{ inches}$$

d. determine Q_{P3}

$$Q_{P3} = (38{,}200\,cfs)\left(1 - \frac{0.153''}{9.5''}\right)$$

$$Q_{P3} = 37{,}600 \text{ cfs}$$

5. STEP 4: Determine surcharge height for Q_{P3} and $STOR_3$

a. from Figure 1 surcharge height for Q_{P3} = 37,600 cfs

surcharge elevation ≈ 767.0'
Top of flashboards = 753.5'
surcharge height = 13.5 feet

Surface area at 767.0 = 72 acres

b. determine $STOR_3$

$$STOR_3 = \frac{\left[(6.5\,ft)\left(\frac{18.2\,ac + 37\,ac}{2}\right) + (7.0\,ft)\left(\frac{37\,ac + 72\,ac}{2}\right)\right](12''/ft)}{(71\,sq.mi)(640\,ac/sq.mi)}$$

$STOR_3$ = 0.148 inches

c. determine $STOR_{AVG}$

$$STOR_{AVG} = \frac{0.153 \text{ in.} + 0.148 \text{ in}}{2}$$

$STOR_{AVG}$ = 0.151 inches

$STOR_3$ and $STOR_{AVG}$ agree to within 2%, therefore accept test flood discharge equal to 37,600 cfs at surcharge elevation equal to 767.0 feet

6. In Conclusion

a. Test flood discharge = 37,600 cfs will overtop top of flashboards by 13.5 feet, the overflow section weir crest by 13.0 feet, and the top of the dam by 6.7 feet

b. spillway capacity -

(1) water surface at crest of dam - elevation 760.3 ft

(a) flashboards in place - (crest elevation = 753.5 ft)

$$Q = (3.65)(34 \text{ ft})(760.3' - 753.5')^{3/2} \approx 2,200 \text{ cfs}$$

(b) flashboards removed - (crest elevation = 751.3 ft)

$$Q = (3.3)(34 \text{ ft})(760.3' - 751.3')^{3/2} \approx 3,030 \text{ cfs}$$

(2) water surface at test flood elevation - ≈ 767.0 ft

(a) flashboards in place

$$Q = (4.03)(34\ ft)(767.0' - 753.5')^{3/2} \approx 6,800 cfs$$

(b) flashboards removed

$$Q = (3.3)(34\ ft)(767.0' - 751.3)^{3/2} \approx 6,980 cfs$$

c. Capacity of sluice gate

(1) water surface at crest of overflow section - elevation 754.0 ft

$$Q = (0.6)(4.5')(4.0')\left[(2)(32.2)(754.0' - 747.7')\right]^{1/2} \approx 220 cfs$$

(2) water surface at top of dam - elevation 760.3 ft

$$Q = (0.6)(4.5')(4.0')\left[(2)(32.2)(760.3' - 747.7')\right]^{1/2} \approx 310 cfs$$

(3) water surface at test flood elevation - 767.0 ft

$$Q = (0.6)(4.5')(4.0')\left[(2)(32.2)(767.0' - 747.7')\right]^{1/2} \approx 380 cfs$$

d. overflow section capacity

(1) water surface at crest of dam - elevation 760.3 ft

$$Q = (3.3)(102\ ft)(760.3' - \underbrace{754.1'}_{\text{avg crest elev.}})^{3/2} \approx 5,200 cfs$$

(2) water surface at test flood elevation - 767.0 ft

$$Q = (3.3)(102\ ft)(767.0' + 754.1')^{3/2} \approx 15,600 cfs$$

III. Using "Rule of Thumb" Guidance for Estimating Downstream Dam Failure Hydrographs examine impact of dam failure

1. Pertinent Data

a. Failure occurs when reservoir level a crest of dam - elevation = 760.3 feet

A. Reach 1

1. STEP 1: Determine reservoir storage at time of failure from previous calcs. Storage = 315 acre-ft

2. STEP 2: Determine Peak Failure Outflow Q_{P1}

a. $Q_{P1} = (8/27)\, W_b \sqrt{g}\, Y_o^{3/2}$

where: W_b = Breach width (use 40% of total length)
= (267 feet)(0.40) ≈ 107 feet
USE 102 feet (see discussion below in A.2.b) - corresponds to length of overflow section

Y_o = Total height from channel bed to pool level at failure

Top of dam - 760.3'
Streambed at toe - 740.0'
20.3 feet

≈ 20.3 feet

$Q_{P1} = (8/27)(102\text{ feet})(32.2)^{1/2}(20.3)^{3/2}$

Q_{P1} = 15,700 cfs

b. Since discharge over the unfailed portion of the dam is significant must add this to the failure discharge

(1) breach width taken to correspond to length of overflow section, since this is close to 40% of dam length

(2) Discharge over dam with water surface at dam crest ≈ 8,600 cfs. If subtract discharge over overflow section (with water at dam crest) from 8600 cfs the remainder will be the discharge over unfailed portion or 8,600 cfs - 5200 cfs equals 3400 cfs.

Therefore $Q_{P1\,(TOTAL)}$ = 15,700 cfs + 3,400 cfs = 19,100 cfs

3. STEP 3 : Prepare stage-discharge curve for Reach 1

a. Pertinent Data

(1) Reach length = 2,600 feet

(2) Channel slope = 0.0044

(3) Manning n = 0.05

(4) Channel shape - ~~trapezoidal~~

(5) Base width ≈ 50 feet

b. See Figure 3 for stage-discharge curve

4. STEP 4 : Estimate Reach Outflow

a. Determine stage for Q_{P_1} = 19,100 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 4.4 feet (Total Stage = 13.2 ft)
above prefailure stage (see pp 29 + 30)

(2) Volume in reach = (reach length) (cross-sectional area of channel)
(above prefailure stage)

$$\text{X-area} = (0.5)(4.4\ ft)(205\ ft + 260\ ft)$$
$$= 1023\ ft^2$$

$$\text{Volume} = V_1 = \frac{(1023\ ft^2)(2600\ ft)}{43{,}560\ ft^2/acre}$$

$$= 61.1\ acre\text{-}ft.$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P2(TRIAL)}$

$$Q_{P2(TRIAL)} = Q_{P_1}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P2(TRIAL)} = (19{,}100\ cfs)\left(1 - \frac{61.1\ acre\text{-}ft}{315\ acre\text{-}ft}\right)$$

$$Q_{P2(TRIAL)} \approx 15{,}400\ cfs$$

c. Compute V_2 using $Q_{P2(TRIAL)}$

From Figure 3 determine stage for $Q_{P2(TRIAL)}$

Stage = 3.0 feet above prefailure stage (Total Stage = 11.8 ft)

X-area = (0.5) (3.0 ft) (205 ft + 245 ft)
= 675 ft²

$$V_2 = \frac{(675\ ft^2)(2{,}600\ ft)}{43{,}560\ ft^2/acre}$$

V_2 = 40.3 acre-ft above prefailure stage

d. Average V_1 and V_2 and compute Q_{P2}

(1) $$V_{avg} = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{61.1\ acre\text{-}ft + 40.3\ acre\text{-}ft}{2}$$

V_{avg} = 50.7 acre-ft

(2) $$Q_{P2} = Q_{P1}\left(1 - \frac{V_{avg}}{S}\right)$$

$$Q_{P2} = (19{,}100\ cfs)\left(1 - \frac{50.7}{315}\right)$$

$Q_{P2} \approx 16{,}000$ cfs

SIEIA CONSULTANTS INC.
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BOSTON, MASS.
ROCHESTER, N.H.

CLIENT Army Corps
PROJECT Moore Mills Dam
DETAIL Hydrologic Calcs.

JOB No. 294-7901
COMPTD. BY BWP
CK'D. BY KMS

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B. Reach 2

1. STEP 3: Prepare stage-discharge curve for Reach

a. Pertinent Data

(1) Reach length = 1,900 feet
(2) Channel slope = 0.0044
(3) Manning n = 0.05
(4) Channel shape - trapezoidal
(5) Base width ≈ 50 feet

b. See Figure 3 for stage-discharge curve

2. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P2} = 16,000 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) above prefailure stage = 1.5 feet (Total Stage 6.9 ft)

(2) Volume in reach above prefailure stage = (reach length) (cross-sectional area of channel)

$$\text{X-area} = (0.5)(1.5\text{ ft})(750\text{ ft} + 945\text{ ft}) \approx 1271\text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(1271\text{ ft}^2)(1{,}900\text{ ft})}{43{,}560\text{ ft}^2/\text{acre}} = 55.4\text{ acre-ft}$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P3(TRIAL)}$

$$Q_{P3(TRIAL)} = Q_{P2}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P3(TRIAL)} = (16{,}000\text{ cfs})\left(1 - \frac{55.4}{315}\right)$$

$$Q_{P3(TRIAL)} \approx 13{,}200\text{ cfs}$$

c. Compute V_2 using $Q_{P3(TRIAL)}$

From Figure 3 determine stage for $Q_{P3(TRIAL)}$

Stage = 1.0 foot above prefailure stage (Total Stage = 6.4 feet)

X-area = $(0.5)(1.0\text{ ft})(750\text{ ft} + 880\text{ ft})$
$= 815\text{ ft}^2$

$$V_2 = \frac{(815\text{ ft}^2)(1{,}900\text{ ft})}{43{,}560\text{ ft}^2/\text{acre}}$$

V_2 = 35.5 acre-ft above prefailure stage

d. Average V_1 and V_2 and compute Q_{P3}

(1) $$V_{avg} = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{55.4\text{ acre-ft} + 35.5\text{ acre-ft}}{2}$$

V_{avg} = 45.5 acre-ft

(2) $$Q_{P3} = Q_{P2}\left(1 - \frac{V_{avg}}{S}\right)$$

$$Q_{P3} = (16{,}000\text{ cfs})\left(1 - \frac{45.5}{315}\right)$$

Q_{P3} = 13,700 cfs

C. Reach 3

1. STEP 3: Prepare stage-discharge curve for Reach 1

a. Pertinent Data

(1) Reach length = 1000 feet
(2) Channel slope = 0.0016
(3) Manning n = 0.05
(4) Channel shape - trapezoidal
(5) Base width ≈ 100 feet

b. See Figure 3 for stage-discharge curve

2. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P3} = 13,700 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 2.5 feet (Total Stage = 12.7 ft)
above prefailure stage

(2) Volume in reach = (reach length) (cross-sectional area of channel)
above prefailure stage

$$\text{X-area} = (0.5)(2.5 \text{ feet})(295 \text{ ft} + 345 \text{ ft})$$
$$= 800 \text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(800 \text{ ft}^2)(1000 \text{ ft})}{43{,}560 \text{ ft}^2/\text{acre}}$$

$$= 18.4 \text{ acre-ft}$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P4(TRIAL)}$

$$Q_{P4(TRIAL)} = Q_{P3}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P4(TRIAL)} = (13{,}700 \text{ cfs})\left(1 - \frac{18.4}{315}\right)$$

$$Q_{P4(TRIAL)} = 12{,}900 \text{ cfs}$$

c. Compute V_2 using $Q_{P4(TRIAL)}$

From Figure 3 determine stage for $Q_{P4(TRIAL)}$

Stage = 2.2 feet above prefailure stage (Total Stage = 12.4 ft)

$$X\text{-area} = (0.5)(2.2\ ft)(295\ ft + 340\ ft) \approx 699\ ft^2$$

$$V_2 = \frac{(699\ ft^2)(1000\ ft)}{43{,}560\ ft^2/acre}$$

V_2 = 16.0 acre-ft above prefailure stage

d. Average V_1 and V_2 and compute Q_{P4}

(1) $$V_{avg} = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{18.4\ ac\text{-}ft + 16.0\ ac\text{-}ft}{2}$$

$$V_{avg} = 17.2\ acre\text{-}ft$$

(2) $$Q_{P4} = Q_{P3}\left(1 - \frac{V_{avg}}{S}\right)$$

$$Q_{P4} = (13{,}700\ cfs)\left(1 - \frac{17.2}{315}\right)$$

$$Q_{P4} \approx 13{,}000\ cfs$$

D. Reach 4

1. STEP 3: Prepare stage-discharge curve for Reach

a. Pertinent Data

(1) Reach length = 2,000 feet
(2) Channel slope = 0.0016
(3) Manning n = 0.05
(4) Channel shape - trapezoidal
(5) Base width ≈ 100 feet

b. See Figure 3 for stage-discharge curve

2. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P4} = 13,000 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 2.2 feet above prefailure stage (Total Stage = 12.4 ft)

(2) Volume in reach above pre failure stage = (reach length) (cross-sectional area of channel)

$$\text{X-area} = (0.5)(2.2\text{ ft})(295\text{ ft} + 340\text{ ft}) \approx 699\text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(699\text{ ft}^2)(2000\text{ ft})}{43{,}560\text{ ft}^2/\text{acre}}$$

$$= 32.1\text{ acre-ft}$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P5(TRIAL)}$

$$Q_{P5(TRIAL)} = Q_{P4}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P5(TRIAL)} = (13{,}000\text{ cfs})\left(1 - \frac{32.1}{315}\right)$$

$$Q_{P5(TRIAL)} = 11{,}700\text{ cfs}$$

c. Compute V_2 using $Q_{P5(TRIAL)}$

From Figure 3 determine stage for $Q_{P5(TRIAL)}$

Stage = 1.6 feet above prefailure stage (Total Stage = 11.8 feet)

X-area = (0.5)(1.6 feet)(295 ft + 325 ft)
≈ 496 ft²

$$V_2 = \frac{(496\ ft^2)(2{,}000\ ft)}{43{,}560\ ft^2/acre}$$

V_2 = 22.8 acre-ft above prefailure stage

d. Average V_1 and V_2 and compute Q_{P5}

(1) $$V_{avg} = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{32.1\ ac\text{-}ft + 22.8\ ac\text{-}ft}{2}$$

V_{avg} = 27.5 acre-ft

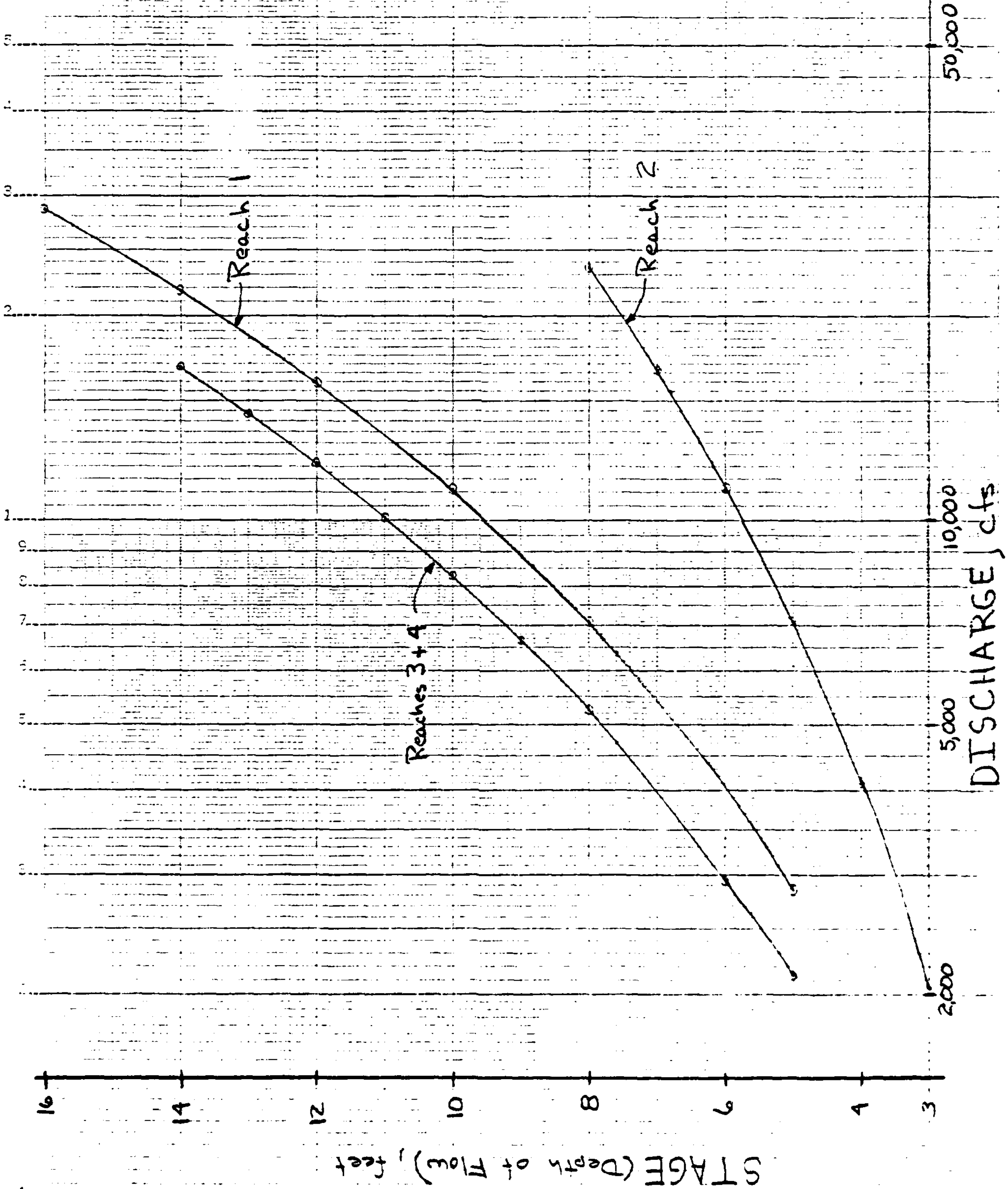
(2) $$Q_{P5} = Q_{P4}\left(1 - \frac{V_{avg}}{S}\right)$$

$$Q_{P5} = (13{,}000\ cfs)\left(1 - \frac{27.5}{315}\right)$$

Q_{P5} = 11,900 cfs

FIGURE 3

DISCHARGE vs STAGE



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Noone Mills Dam

Job # 274-7901

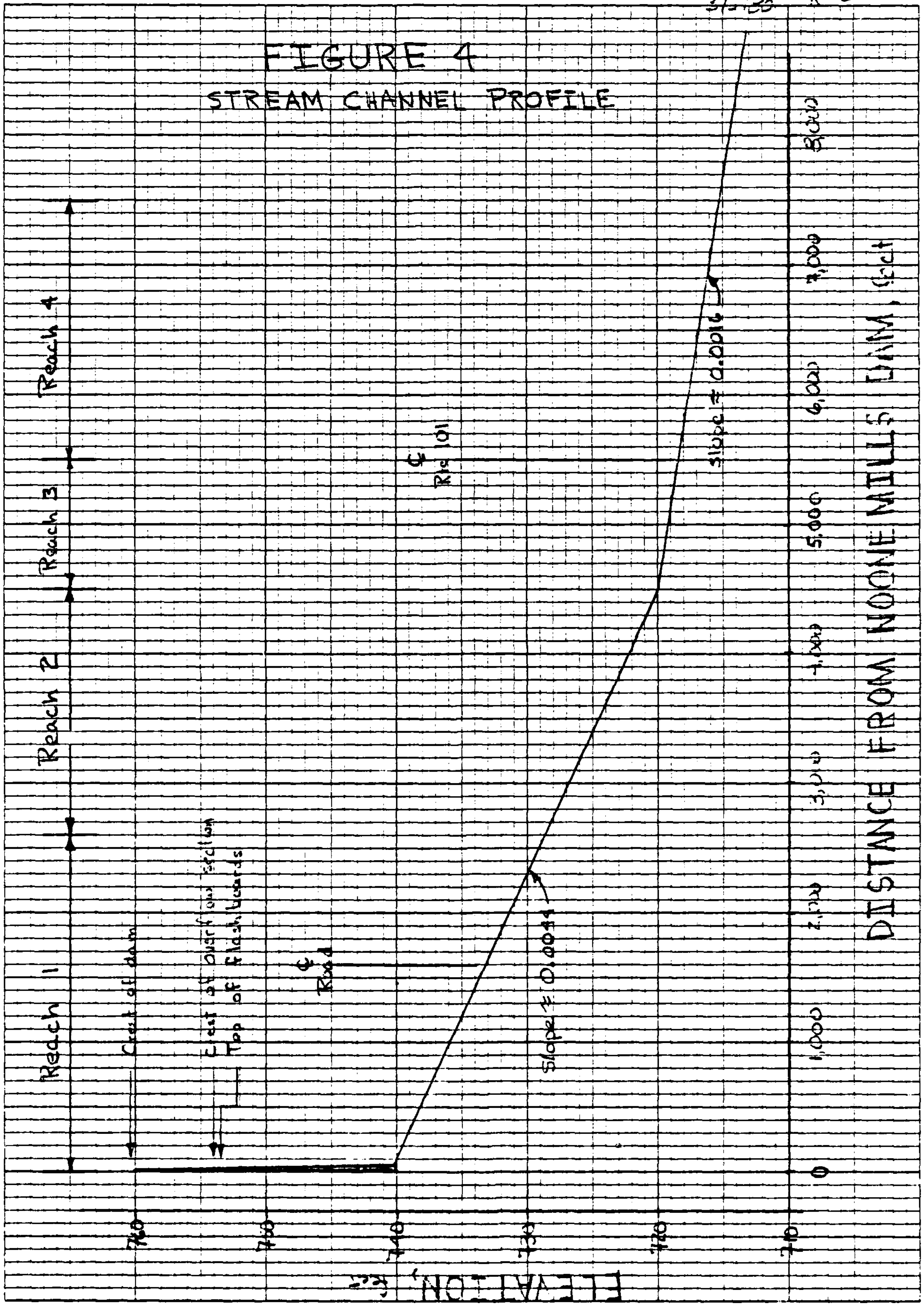

FIGURE 4
STREAM CHANNEL PROFILE

IV Compare dam failure discharge stage to prefailure stage resulting from discharge over spillway with water surface at the crest of dam (elevation 760.3 feet)

A. Since overflow section and spillway are quite long, the discharge over the dam will be quite large. Therefore, the stage resulting from the discharge over the dam (with water surface at crest of dam) must be compared to the stages for the failure discharge examined in Section III

1. From Figure 1 the prefailure discharge over the dam was estimated to be 8,600 cfs

2. Compare stages for various reaches using Figure 3

a. Compare stages in Reach 1

(1) Stage for Q_{P2} = 16,000 cfs is 12.0 feet

(2) Stage for 8,600 cfs is 8.8 feet (prefailure tailwater)

b. Compare stages in Reach 2

(1) Stage for Q_{P3} = 13,700 cfs is 6.5 feet

(2) stage for 8,600 cfs is 5.4 feet (prefailure tailwater

c. Compare stages in Reach 3

(1) stage for Q_{P4} = 13,000 cfs is 12.4 feet

(2) stage for 8,600 cfs is 10.2 feet (prefailure tailwater)

d Compare stages in Reach 4

(1) stage for Q_{P5} = 11,900 cfs is 11.9 feet

(2) stage for 8,600 cfs is 10.2 feet (prefailure tailwater

3. Conclusions

a. The stages resulting from the failure discharge are about 2 to 3 feet higher than those associated with the prefailure tailwater in Reaches 1 and 3, which are the two reaches with the greatest hazard potential. It also appears that this increase in stage would result in a greater hazard than that for the prefailure tailwater.

NOT AVAILABLE AT THIS TIME

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